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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

FLOOD CONTROL WITH SPECIAL REFERENCE TO THE MISSISSIPPI RIVER

A SYMPOSIUM*

NOTE.—Discussion on this Symposium will be closed in **April, 1928.**

* Presented at the Technical Meeting and at the meeting of the Waterways Division, Columbus, Ohio, October 12 and 13, 1927. Papers by Lt.-Col. George R. Spaulding, Corps of Engrs., U. S. A., and by S. M. Woodward and Floyd A. Nagler, Members, Am. Soc. C. E., presented at these meetings, are not included in this number of *Proceedings*.

FLOOD CONTROL WITH SPECIAL REFERENCE TO THE MISSISSIPPI RIVER

INTRODUCTION

By Edgar Jadwin,* M. Am. Soc. C. E.

The importance of controlling the floods of the Mississippi River is now generally recognized. It is a National problem. The Mississippi Valley contains the largest area of rich soil in the country. It must be inhabited and used to as great an extent as practicable.

This valley is about 600 miles long and 50 miles wide. The entire area is generally considered to have been formed by the river itself. It is the natural bed of the river in time of flood. In its natural state it was flooded practically annually. This rendered intensive cultivation of the land too uncertain for practical purposes.

It has been compressed or narrowed in recent years to an increasing extent. This work has been done by levees, which have kept the water off the land so that it can be cultivated on an average of at least four out of five years. Some parts of the reclaimed land have not been flooded for much longer periods.

How much further can the river be compressed without having it break out at intervals and do even greater damage than it would have done if the works were not constructed? The population in the valley is increasing and a crevasse is now of greater threat than formerly. In the further control of the river, to what extent shall levees, dredging, reservoirs, spillways, floodways, and other safety measures be used? Will it be possible to reclaim the entire valley permanently? Or must some part of it be permanently or intermittently dedicated to the service of the river?

Every one has probably commented in watching some piece of machinery, such as the steam shovel, upon its likeness to a living thing. The steam shovel is guided by a man, expresses his aim, and has good reasons for appearing lifelike. The Mississippi River, without human aid, comes as near being lifelike in its own right as any inanimate object in the world. It consists of water and dirt aided by gravity. Some one has calculated that its volume in flood would, in two weeks, fill the great Lake Erie; some one else, that its energy when in flood would, if expressed in pounds and the square of its velocity, be measured in quintrillions. It tears the dirt from its banks and bed. Some one else has calculated the dirt carried annually as equivalent to a cubic mile in volume. A large part of this dirt was formerly deposited on the alluvial valley, re-fertilizing the ground and slightly raising it. The works along the river and at its mouth so concentrate the flow of this dirt that the mouth of the river is being extended into the Gulf of Mexico 5 miles per 100 years.

The speaker has just returned from an inspection of a part of the territory in the valley. In talking with those who have given the best part of their

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lives to the river, he is impressed with their admission of the results obtained by levees and equally by the shake of the head when there is talk of raising them much higher. They know the power of the river, and they know that work to be successful must reckon with this power. They know the danger of the crevasse and that the higher the flood-plane and the greater the population behind it, the greater the threat of the crevasse. The first three papers presented herewith largely furnish data on the history of the river, the work done on it, and fundamental data needed for the solution, closing with an excellent picture of the flood and its rescue work.* In the other papers, various phases of the problem will be considered. They may not all agree, but if one will follow through, he cannot fail to carry away a greater and broader comprehension of the subject.

* Refers to an illustrated address by Lt.-Col. George R. Spalding, Corps of Engrs., U. S. A., which is not included in this number of *Proceedings*.

RESUME OF THE MISSISSIPPI RIVER FLOOD PROBLEM

BY C. McD. TOWNSEND,* M. AM. SOC. C. E.

INTRODUCTION

The problem of flood protection is one which has vexed mankind for ages. In Genesis VI, 12, 20, 24, a flood is described, which was caused by a rainfall of 40 days and 40 nights and which raised the waters 15 cubits, covered the earth for 150 days, and destroyed all animal life in its path. This description of the Deluge would not be a very exaggerated account of the flood of 1927 in the Mississippi Valley. Moreover, there is an analogy between the Ark that Noah constructed as a means of flood protection and the boats and barges used by Secretary Hoover for the same purpose.

The Chinese appear to have records of floods extending over a period of 4 000 years, and "thirty-five years ago more than 1 000 000 people (some say 7 000 000) perished by drowning and starvation, resulting from one flood which came from a break through the south dike of the Yellow River, about 20 miles above Kai-feng City"† The annals of the countries of Europe also record disastrous overflows with a loss of life far greater than that from any which have occurred in the United States. The loss of life from floods in Europe during the past three months largely exceeds the deaths in the United States from the same cause during the past year.

The most fertile portions of the earth's surface are the valleys of streams that are periodically subject to overflow, and, therefore, enriched by the alluvium brought down and deposited from the mountains. This fertility induces a settlement of the country, and the farmer who first tills the valley is prepared to lose an occasional crop by flood, relying on the increased yield in other years to recoup him for his losses. In fact, in certain regions, the difference in yield between hill land and bottom-land is so great that the farmer reaps a greater profit by tilling the bottoms, even if he loses his crop every third or fourth year, as is demonstrated by the high prices that are paid for land along the banks of the Missouri River even when unprotected by levees.

The early settlers usually adopted Noah's method of flood protection. They built their huts on the high ground along the river bank, and constructed a boat into which they moved with their few household goods whenever the waters invaded their dwellings. It is the method used by the trappers and lumbermen living along the bayous in the Atchafalaya Basin at the present time.

A second method of flood protection adopted at an early date was that of building a mound of earth or of erecting a platform above the flood waters,

* Colonel, U. S. A. (*Retired*), Washington, D. C.

† "Flood Problems in China," by John R. Freeman, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1405.

on which the home was located. The numerous Indian mounds found in the Mississippi Valley were probably built for that purpose.

If it is only necessary to provide for the population, this method of flood protection is economical and efficacious, and is adapted to rivers which overflow their banks at periods of the year when vegetation is not injuriously affected thereby. Thus, in the Valley of the Lower Nile, irrigation is necessary to produce a crop. It is only after flood waters have covered the land that vegetation will grow. Its floods are caused by rains originating in the tropics, and occur early in the spring before the period of planting. The problem in Egypt is not to prevent floods, but to impound their surplus waters so as to utilize them for a further flooding of the land during periods of drought while the crops are growing. In the Lower Mississippi Valley conditions are reversed. The floods caused by rains falling in the temperate zone through which the river flows, inundate the valley after the crops begin to mature, and they will be frequently destroyed unless protected from the overflow.

The occasional loss of a crop or even the destruction of his habitation was of little consequence to the aborigine, but as civilization progresses man becomes more firmly bound to the soil. It was a natural development from the mound of earth on which the dwelling was placed to a ridge of earth surrounding the area that was being cultivated, and so long as the area of land which was being tilled was a small percentage of that of the valley, levees could be constructed cheaply by the property owner, which would protect his crops from ordinary floods. If he was energetic and far-sighted, he insured his property from even the occasional overflows, which overwhelmed the remainder of the valley, by building his levee a little higher than that of his neighbor.

As the population increased and a larger proportion of the river bed was used for agricultural purposes, a mysterious increase in the heights and frequency of floods was observed, which was attributed by scientists in olden days to the wrath of the gods and by those of the present generation to the cutting down of forests which are assumed to have heretofore prevented rainfall or held in place soil which is now raising the river bed. A much simpler solution of the mystery occurs to the engineer, that is, that the same quantity of water can only be made to flow through a contracted channel by increasing the head or height and thus creating a greater velocity.

In narrow river valleys with small flood discharges, this increase in flood heights is not excessive, but when it is attempted to remove from the Mississippi River about 30 000 sq. miles of its flood channel, and contract a flow over an original width exceeding 40 miles to a channel less than 1 mile wide, the problem is a serious one. The nations of Europe have solved the problem for such rivers as the Rhine, Seine, Loire, Rhone, Po, and the Danube under the conditions which exist in that region, but it is to be noted that the flood discharge of any of these rivers is approximately that of a large crevasse during the 1927 Mississippi River flood.

The Legal Status of Flood Protection

When the French under Bienvielle settled the Lower Mississippi Valley, they had a knowledge of the methods of flood protection then adopted in Europe, and the original grants of land contained a proviso which required the grantee to construct and maintain a levee line along the river-front of his property. This servitude on the land was maintained in Louisiana after it was admitted into the Union, and only ceased by reason of an amendment adopted by the State within the past five years, which transfers it to the levee district in which the land is located.

This servitude was not oppressive so long as the plantations to be protected were limited to those in the State along the river bank, as only an insignificant part of the river bed was being occupied. It was only after settlement extended to the neighboring States of Arkansas and Mississippi that an appreciable increase in floods was observed. The lands of these States were originally a part of the public domain, but the "Swamp Act" passed by Congress, September 28, 1850, ceded those subject to overflow to the States in which they were situated, with a proviso that the cession to the State was to enable it "to construct the necessary levees and drains to reclaim the swamp and overflowed lands therein." The States of Louisiana, Arkansas, Mississippi, and Missouri organized offices for the sale of the swamp lands, and appointed commissioners for the location and construction of levees.

There had already been an appreciable increase in flood heights, which led to the passage of an Act approved September 30, 1850, directing "a topographical and hydrographical survey of the Delta of the Mississippi River with such investigations as may lead to determine the most practicable plan for securing it from inundation", etc. The results of this survey were published in the report of Humphreys and Abbot on "The Physics and Hydraulics of the Mississippi River." This immortal work contains a scientific analysis of nearly every suggestion of flood relief which is being advocated at the present time, and when the limited data which were then available are taken into consideration, it is a marvelously accurate forecast of the work necessary to protect the valley securely. Fig. 1 shows the average dimensions of the levee that existed in the vicinity of Arkansas City, Ark., in 1882, twenty-one years after they submitted their report, the levee profile recommended by them, and that suggested by the writer in 1893, and practically adopted by the Mississippi River Commission in 1913.

During the Civil War, the levee lines were cut by the contending forces, and seriously damaged by floods. After the Civil War local levee boards generally replaced the State organizations, originally appointed under the Swamp Act, although in Louisiana, a State Board of Engineers continued to exercise supervision over the local levee boards which had replaced the riparian owner in levee construction. Although the construction of levees by local boards gave more satisfactory results, than when the riparian owner was the sole judge of the dimensions, this method of construction was also objectionable. While one district was constructing levees of relatively large size, a neighboring district, either from a lack of funds or from erroneous

views of the height which floods would attain, would provide a weaker section, and crevasses would occur in levees beyond the jurisdiction of the more provident residents, which would overflow their lands.

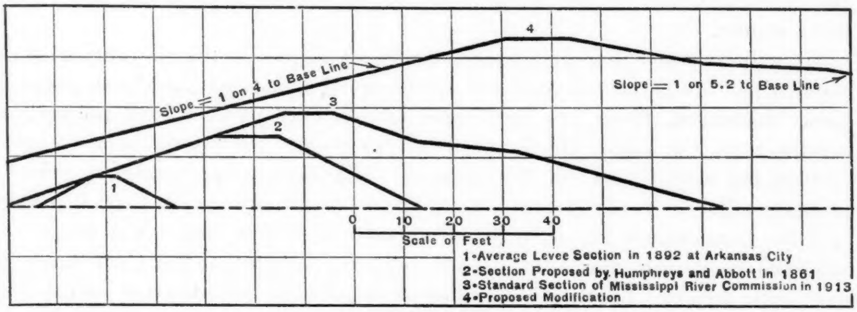

Fig. 1.

By an Act of Congress approved June 22, 1874, a commission was appointed to "make a full report to the President of the best system for reclamation and redemption of said alluvial lands from inundation." By an Act approved June 28, 1879, the Mississippi River Commission was created, and the duty of completing the survey authorized by the Act of June 22, 1874, was transferred to that body, and it was also directed "to take into consideration and mature such plan or plans, and estimates as will correct, permanently locate and deepen the channel, and protect the banks of the Mississippi River; improve and give safety and ease to the navigation thereof, and facilitate commerce, trade and the postal service." The Act also contained a proviso that "the Commission shall report upon the practicability, feasibility and probable cost of the various plans known as the jetty system, the levee system, and the outlet system, as well as upon such others as they deem necessary."

The Act of March 3, 1881, making appropriation for the work of the Commission contained, however, the proviso that "no portion of the sum hereby appropriated shall be used in the repair or construction of levees for the purpose of preventing injury to lands by overflow, or for any purpose whatever except as a means of deepening or improving the channel of said river."

The Act of March 13, 1892, modified the wording of this proviso and directed that the funds appropriated "shall be expended for the general improvement of the river, for the building of levees and for surveys, * * * in such manner as in their opinion shall best improve navigation and promote the interests of commerce at all stages of the river."

The Act of March 1, 1917, contains a proviso that "no money appropriated under the authority of this act shall be expended in the construction or repair of any levee unless and until assurances have been given satisfactory to the Commission that local interests protected thereby will contribute for such construction and repair a sum which the Commission shall determine to be just and equitable, but shall not be less than one-half of such sum as

may have been allotted by the Commission for such work." The same Act extended the jurisdiction of the Commission over levee construction to Rock Island, Ill. The Act of March 4, 1923, further extended its jurisdiction to the tributaries and outlets of the river so far as they may be affected by its flood waters.

It will be noted that in all these Acts of Congress while the authority of the United States over levees and its financial assistance has been progressively increased, there has been considerable care exercised to preserve a servitude on the lands which are to be benefited. As Comptroller General McCarl has recently stated, "the primary obligation for protection of private property adjacent to the streams for which co-operation between local interests and the Government has been provided, rests upon the individual States or subdivisions within which the private property is located, the Government's part being to assist financially in connection with the maintenance and development of the river as a navigable stream only in proportion to the amount made available by such State or subdivision."

Under this construction of the law, the question of servitude becomes the most important problem which Congress must consider in any project for an extensive development of flood protection. The States have transferred it from the riparian owner to the various levee districts. Shall it now be assumed by the State or by the Federal Government? It is evident from the results of the 1927 flood, that the cost of any project for absolute security will far exceed the capacity of the riparian owners to meet their portion of the expense provided by existing law. It is also apparent that the farmer tilling the less fertile hill lands of Arkansas and Mississippi will never consent to an increased taxation for the purpose of further increasing the value of the plantations in the delta region, and insuring the maintenance of such values.

The Mississippi River Commission

It will be noted that while Congress in the Act creating the Commission directs it to investigate various methods of flood protection, yet in the Act appropriating funds to carry on its work, it is expressly prohibited from using them for the purpose of protecting land from overflow. This apparent inconsistency in legislation had its effect on executive action and, therefore, merits explanation.

Most levee engineers in the Mississippi Valley had not accepted the conclusions of Humphreys and Abbot that levee construction would largely increase flood heights. There was a general belief that the concentration of the flood discharge would enlarge the river bed and that this enlargement would materially reduce the levee grade, and would greatly improve low-water navigation. These views had Congressional support, and as General Humphreys was then Chief of the Corps of Engineers, U. S. Army, it was considered advisable to appoint a commission of which the majority would not be connected with that Corps, and thus insure a more liberal policy in determining the effect of levees on channel improvement. As a majority of Congress

did not approve of appropriating public funds for the protection of private property, it was considered advisable to limit the powers of the Commission thus created by the proviso of the Act of 1881.

As a majority of the Commission which was first appointed believed that levees would improve low-water navigation, a liberal allotment of funds for levee construction was assured, but the proviso mentioned rendered it necessary that the levees thus improved should be located where they would produce the greatest influence on the low-water channel, instead of where they would best preserve the land from overflow. As low-water navigation was notoriously bad in the upper reaches of the river, there was a tendency to concentrate the allotments for levee construction along those sections, and particularly in the sparsely populated levee districts where the inhabitants did not have sufficient funds to protect themselves. A great impetus was given to levee construction, and a marked increase in flood heights resulted, so that not infrequently the protection afforded to the weaker districts caused crevasses in those more densely populated.

Levee construction from 1881 to 1891 not only largely increased flood heights, but it also failed to produce any appreciable improvement in low-water navigation. The Commission then investigated the effect of dredging on river bars, and came to the conclusion that a low-water navigation of 9 ft. could be obtained temporarily by this means and could be maintained more economically than by either levee construction or bank revetment. Congress then authorized the construction of dredges for that purpose, and permitted the allotment of funds for levee construction whenever it would facilitate high-water navigation of the river. Federal funds were allotted for levee enlargement under this proviso, and the weaker levee districts continued to derive a greater relative benefit.

The Act of 1917 caused a radical change in the methods of making allotments by providing that at least one-third the cost of levees constructed by the United States should be borne by local authorities. Under this method of making appropriation the more populous districts became the greater beneficiaries, as they could more readily raise the funds required to obtain Federal assistance. The weaker districts had difficulty in keeping pace with their more fortunate neighbors. The result is clearly shown in the record of crevasses during the recent flood, which most vividly emphasizes the Biblical saying that "unto every one that hath shall be given, and he shall have abundance; but from him that hath not shall be taken away even that which he hath." The levee districts which escaped overflow owe their immunity to the poverty of their less fortunate neighbors.

In compliance with the provisions of the organic act, the Commission has made extensive surveys and investigations for the purpose of determining the practicability, feasibility, and relative cost of the various methods of flood protection that have been proposed. The results of its investigations are scattered through its annual reports for the past 47 years, and have been summarized in papers and reports submitted by the writer from time to time. The gist of its conclusions was invariably that levees afforded the most prac-

tical, feasible, and economic method, although its process of reasoning varied with changes in the membership of the Commission.

So long as a majority believed that only an increase of flood height of 3 ft. should be provided for, which was the provisional grade for many years, the question was hardly worthy of serious argument. The crudest estimates of cost conclusively demonstrated that levees were so much cheaper than either reservoirs, the diversion of tributaries, or spillways, that a further discussion of the practicability or feasibility of these methods was merely an academic compliance with the law. It was only after a majority of the Commission endorsed Humphreys' and Abbot's views, and attempted to make provision for the greatest flood which had then occurred, that a careful analysis of the various methods of flood control became imperative.

After the flood of 1912, such an attempt was made. The Commission assumed as a major premise that the flood of 1882 was the greatest to which the levee system would be exposed, but based its estimates on the flood of 1912 which, while of shorter duration, had the same estimated maximum discharge, and had been more accurately gauged and measured. The Commission unanimously agreed that levees were the most economical method of flood protection, and that reservoirs were impracticable; but there was such a diversity of opinion with regard to the feasibility of spillways and the diversion of tributaries that the discussion of these subjects was omitted from the report. The views of individual members, however, were printed as Commission documents.

Effect of an Increase in Discharge on the Conclusions

When Humphreys and Abbot submitted their project for the protection of the Mississippi Valley from overflow, they based their computations on the flood of 1858 which was the largest flood through the entire valley of which there was then any record, although the flood of 1844 exceeded it above Cairo, Ill., and that of 1785 had similar characteristics to that of 1844. As stated, the Mississippi River Commission based its estimates of levee grades on the flood discharge of 1882, which largely exceeded that of 1858, and has only been exceeded since 1785 by that of the flood of 1927.

While both Humphreys and Abbot and the Mississippi River Commission came to the conclusion that levees were the most practicable and most economical method of flood protection, they based their argument on an assumed flood discharge, and it by no means follows that such flood discharges as are now being discussed as possibilities can be more cheaply controlled by this method. Any project which contemplates increasing the heights of existing levees at certain localities about 15 ft., not only affects the item of cost, but raises the question of the practicability and feasibility of maintaining such a levee line after it has been constructed. Whenever the Mississippi River flows in a channel 30 or 40 ft. deep, it has a powerful erosive force, and to expose levees in concave bends or across points to such a force would invite their destruction.

The cost of the existing methods of levee construction increases as the square of the height, and if absolute protection is demanded for such levees as indicated, the cost over existing methods will probably approach the cube of the height. The cost of a weir is independent of the height of the water flowing over it, and the cost of a dam is approximately a direct function of the quantity of water which it impounds. If possibilities instead of probabilities are to control future levee construction, the cost will become so enormous that the economic solution of the problem will be found in other methods of flood protection.

The problem of flood control by reservoirs has been much simplified by the record of rainfall during the flood of 1927. It is self-evident that a reservoir which will reduce flood heights must be constructed in a locality where it can receive the waters which create the flood. This eliminates from consideration all reservoir sites in the Missouri Valley above Kansas City, Mo., in the Mississippi Valley above Rock Island, and in the Ohio Valley above Pittsburgh, Pa., as the rainfall in those regions did not exceed a total of 6 in. during the months of March and April. It practically limits their location to those portions of the States of Illinois and Indiana which drain into the Wabash River, the streams of Southwestern Illinois and Southeastern Missouri which discharge into the Mississippi River below St. Louis, Mo., the drainage area of the Tennessee River below the Wilson Dam, and the rivers of Arkansas, Mississippi, and Louisiana which empty into the river below Cairo. In this area there was a rainfall exceeding 12 in. in 60 days. The prairie country of Illinois, Indiana, and Southeastern Missouri is not adapted to the construction of large reservoirs. The flood of 1927, therefore, clearly demonstrates that the only reservoir or reservoirs capable of controlling the waters which created it must be located in the delta region below Cairo, that is, in the area which it is desired to protect by their construction.

It is theoretically possible to create in either the St. Francis, the Yazoo, or the Tensas Basins, reservoirs which would have sufficient storage capacity to retain the surplus waters of the flood of 1927, but the cost would be excessive. Humphreys and Abbot suggested the utilization of the Tensas Basin to reduce floods, and the Mississippi River Commission has had before it propositions to utilize the St. Francis Basin for the same purpose. In the days of Humphreys and Abbot, the greater portion of the land needed for such a purpose was a part of the public domain and could have been acquired at a nominal cost. When the utilization of the St. Francis Basin was being considered, the State authorities were having difficulty in collecting the nominal taxation levied upon it. Levee construction, however, has enormously increased the value of property in both basins, and it is extremely doubtful whether the area for the reservoirs could be acquired by the Federal Government at the present time for $100 per acre, when the cities and villages which have sprung up in this region are taken into consideration, notwithstanding the reduction in values of real estate which must have resulted from the overflow.

The fact that it is estimated that more than 600 000 were made homeless by this flood and that it is one of the great catastrophes of the century is striking proof of the development of the delta from levee protection. Other floods have overflowed a greater area, but it was then so sparsely settled that the disaster did not create much comment beyond the Valley. If, to the cost of condemning about 6 000 sq. miles of reservoir sites, is added the cost of embankments for retaining the water higher than the levees which would be required, reservoirs can not be recommended as an economic proposition.

With spillways, conditions are reversed. For years, the writer has maintained that the existing Commission grade for levees below the mouth of Old River is the limiting line for their economical use. In a paper on spillways* presented before the Louisiana Engineering Society in 1925, he invited attention to the fact that a reduction in the flow of the Atchafalaya Outlet by the closure of Old River would necessitate the construction of a spillway to compensate for the increased discharge down the main river which would result therefrom. He also advocated the construction of a spillway below New Orleans, La., which would reduce flood heights at that locality, considering it a cheaper solution of the overflow problem for that city than raising its docks and the walls of the Industrial Canal to the Commission grade. He was then discussing the problem of lowering flood heights about 1 ft. The question ceases to be open to discussion, when it is proposed to provide for a discharge in the lower river 30 or 40% greater than that assumed by the Commission.

Although favoring a spillway which spills the water, and allows it to flow unrestrained to the Gulf, the writer also invited attention to the danger which arises from contracting the flow. The spillways required to reduce the discharge now considered possible to that assumed by the Commission, must have a flow exceeding that of the Yellow River in China or that of the Po River in Italy. The proposed flow in the spillway will exceed that of the South Pass of the Mississippi River, and approximately equal that of Southwest Pass, or Pass à Loutre, the other mouths of the river. Both South Pass and Southwest Pass have been successfully deepened by confining their flood discharge between parallel jetties. There are, however, hydraulic engineers who assert that the same discharge can be confined between parallel levees in a spillway without producing scour. It is a curious coincidence that the width proposed after the flood of 1922, for a spillway emptying into Lake Borgne, corresponds to the distance between the shore ends of the jetties at Southwest Pass recommended by noted levee engineers and adopted by Congress as the original project for its deepening to 35 ft., although a Board appointed by the Chief of Engineers in 1916 reduced this width from 6 000 ft. to 2 400 ft. because the excessive width as first approved was unable to maintain a channel depth exceeding 25 ft. The slope in Southwest Pass during floods is about 0.2 ft. per mile, while that in the proposed spillway would have exceeded 1 ft. per mile.

* *Proceedings*, Louisiana Eng. Soc., Vol XI, 1925.

A deep hole is invariably excavated in the contracted section of a crevasse, and the material thus scoured is deposited approximately on the arc of an ellipse enclosing the hole, and at a distance from it determined by the reduction in velocity caused by the dispersion of the flow. This action is similar to that which creates a bar at the mouth of the river. The forces which create the hole at a crevasse will exist in a spillway, and if not reduced by allowing its water to expand will be propagated its entire length. Some accidental cause, as an abandoned ditch or even a line of fence posts, will induce a local scour and when such a channel starts, it will enlarge. By such means both the Po and Yellow Rivers have created channels through which the entire river now flows.

In providing for possibilities which have not yet arisen, it is rash to ignore probabilities which have already occurred. The diversions of the Po and Yellow Rivers are historic facts while the excessive discharges now being discussed may be mere flights of a vivid imagination. The extension of the spillway at Point La Hache a sufficient distance up stream, will produce the same lowering of flood heights at New Orleans as the one proposed at Poydras, and at less cost. It will also reduce to a minimum the danger of river diversion.

An Act of Congress approved April 17, 1926, provides,

"* * * that the Secretary of War be and he is hereby authorized and directed to cause a survey to be made, and estimates of the cost of such controlled and regulated spillway or spillways as may be necessary for the diversion and control of a sufficient volume of the excess flood waters of the Mississippi River between Point Breeze and Fort Jackson in Louisiana in order to prevent the waters of the river exceeding 16, 17, 18, 19 and 20 ft. on the Carrollton gauge at New Orleans, and of approximately 46, 47 and 48 ft. on the gauge at Simmesport on the Atchafalaya Outlet. * * *"

Section 2 of the Act, however, contains a proviso "that no spillway shall be constructed as a result of the survey authorized by this act whereby the waters of the Mississippi River would be diverted into Mississippi Sound". A Board of Engineer Officers has been appointed in accordance with the terms of this Act, and will presumably submit its report before the next session of Congress. The Congress could never have contemplated the occurrence of the flood of 1927 when it passed the Act, and a strict compliance with its provisions under existing conditions would thwart its intent.

The river attained a height of 15.9 ft. on the Carrollton gauge during the flood of 1867, and of 46.3 ft. on the Red River Landing gauge. The levee line had been neglected during the Civil War, and practically annihilated by the floods of 1862 and 1867. A continuous spillway existed on both banks of the river from Baton Rouge, La., to its mouth, with the exception of the area occupied by the City of New Orleans. To assume that any spillway can be constructed in a semi-controlled river having a discharge at least 50% greater than that of 1867 which will only raise the gauge at Carrollton 0.1 ft. is absurd. Moreover, the proviso that no spillway constructed under the law shall divert water into Mississippi Sound precludes the construction of any spillway discharging into Lake Pontchartrain or Lake Borgne, and sug-

gests as a solution of the problem, a further extension up stream of the spillway recently built below Point La Hache, where the escaping waters flow into Breton Sound. The removal of the entire levee line on the east bank of the river below the Lake Borgne Canal could not be expected to reduce flood heights at Carrollton much below 20 ft. with a river discharge as great as has been recently estimated.

The gauge at Simmesport, La., can only be maintained at 46 ft. by closing Old River and preventing the discharge of the Mississippi River from entering it, and even then the existing reservoir at the lower end of the Tensas Basin will have to be maintained to reduce the maximum discharge which it is assumed flowed in the Ouachita and Red Rivers during the 1927 flood. The provisions of the Act which limits the survey to that part of the river between Point Breeze and Fort Jackson, La., also prevents the Board from investigating what gives promise of being the most satisfactory solution of the theoretical problem which is now being considered.

The disastrous results of the flood of 1927 are not primarily due to the floods of the Ohio and Upper Mississippi Rivers, but to the rainfall of 18 to 25 in. which occurred in the Lower Valley during the flood. Early floods in the rivers of Arkansas created such stages in the main river from Helena, Ark., to Vicksburg, Miss., that it destroyed what is termed the river's reservoir capacity, so that later moderate floods from the upper rivers created gauge heights through this section far exceeding any that had been heretofore recorded for the same discharge.

When the flood of 1927 is compared with that of 1912, on which the Commission based its estimates, it is noted that at Helena a discharge 300 000 sec-ft. less in 1927 than in 1912 increases flood heights 2.6 ft.; at Arkansas City, a reduction of 300 000 sec-ft. is accompanied by an increase in flood heights of 5.5 ft.; while, at Vicksburg, the same flood discharge increases flood heights 6.9 ft. When in addition to this disastrous condition of affairs, the crests of the floods of the Arkansas and White Rivers poured their waters on the crest of the main river flood, the question arises whether the cheapest solution of the problem would not be to separate these rivers entirely from the Mississippi and provide a separate channel for them across the Tensas Basin.

The Commission estimated that these rivers would only add to the maximum flood discharge of the Mississippi about 200 000 sec-ft. at their mouths, relying on the reservoir capacity of the main river and the lack of coincidence of the crests of the floods to reduce their reported maximum discharges to this quantity. If provision must be made for adding about 1 000 000 sec-ft. to a river the reservoir capacity of which has been exhausted, the grade of the levee line becomes excessive. If these rivers were diverted the reservoir capacity of the main river would be much greater than that estimated by the Commission and the flood of 1927 would have been propagated through the lower river, without the necessity of building spillways even if Old River was also closed. The cost of the proposed levee enlargement, and of spillways, would largely exceed the cost of such diversion.

It must be admitted, however, that this solution of the flood problem will not reduce flood heights at Simmesport to 46 ft., and that the determination of flood heights along the diversion channel would be a problem difficult of solution for the next twenty years, during which time the diverted flow would be excavating its bed. While the cost of this project may be many hundred million dollars less than that of any other project that will afford complete protection from any possible flood, it is still so great as to raise the question whether the increase in value of the land to be benefited would justify the expenditure.

The practicability of flood control by reforestation, by straightening the river, by enlarging its low-water channel with dredges, and by increasing its reservoir capacity by placing the levee lines at a greater distance from the channel, is not affected by a variation in the discharge. The only one of these propositions that is now being seriously considered is a further increase in the reservoir capacity.

For every mile that the levee line is moved away from the river bank, the reservoir area during floods will be increased by about 1 000 sq. miles, but the area capable of being cultivated will also be reduced by the same amount. The land along the concave banks of the river is the highest land in the delta, the most highly cultivated, and the location selected for the houses and barns of the inhabitants. Many levee districts would be bankrupt by a change of 1 mile in the location of the levee line. A large proportion of the productive land which now pays the taxes, would be left unprotected. While the Commission places levees in bends at a considerable distance from the bank, it is solely to protect them from river caving and insure a levee life of about 20 years.

The practicability of the project also merits consideration. If the flood of 1927 is to be an annual occurrence, it would adjust its channel to its discharge, but if it does not again occur during the next 100 years, the minor intermediate floods will tend to build up the river banks just as they have for ages, and by the time the second flood occurs, this fill will have largely reduced the reservoir depths.

It has also been suggested that complete protection necessitates a change in the standard levee section adopted by the Commission. In a discussion of the paper by the late William Starling, M. Am. Soc. C. E., entitled "The Discharge of the Mississippi River", presented to the Society in 1896,* the writer stated that,

"* * * the forms adopted for the levee section differ materially from those which obtain in reservoir construction. The causes of this difference arise from the impracticability of finding in an alluvial valley the foundation impervious to water that is an essential condition in reservoir construction. The material for constructing the puddled core is also not readily obtainable, and its value is not so evident when it can be flanked by water passing through a permeable foundation into the embankment beyond.

"The essential condition of levee construction is a mound of earth of sufficient height so that water cannot flow over its top; of sufficient mass so

* *Transactions*, Am. Soc. C. E., Vol. XXXV (1896), pp. 341–343.

that surfaces of saturation cannot be formed through it on which the superincumbent material will slide; of sufficient width of base, so that the water flowing through the permeable foundation will not have sufficient force to remove any material, it being assumed that the earth has been placed with care so that there are no channels left across the embankment through which the waters can directly flow. This mound of earth must be protected from the eroding action of rains, and its river surface from erosion by river currents and waves during floods. The grade must be somewhat higher than the highest flood to afford protection from wave action during storms.

"With the extreme variations in permeability which are found in the soils of which levees are constructed, variations in form become necessary, but practical experience with the soils of the Mississippi bottoms has developed certain forms which are of general application. A slope of 1 on 3, when well sodded with Bermuda grass, has been found to resist the action of rain, of river currents and of minor waves. In localities exposed to the full force of waves through long reaches of the river, it is necessary either to reinforce the sod with some more permanent form of revetment, or to adopt a gentler slope.

"A width of crown varying from 6 to 10 ft. generally exists. The width to be given to the crown is interdependent with the slope to be given the land side of the levee. An 8-ft. crown affords convenient space for patrolling the levee line during floods, and storing such material as may be required to repair any damage which may be inflicted by storms. With a width of crown of 8 ft., a land slope of 1 on 2 can be safely employed until the levee attains a height of 6 ft.; with higher levees unless the soil is especially adapted to levee construction, there is a tendency to form a surface of saturation through the mass on which the upper portion is liable to slide. Levees not exceeding 12 to 14 ft. require a land slope of 1 to 3; above these heights in the upper basins of the river, trouble begins to be experienced with the foundation, and still gentler slopes are necessary, or the same result can be attained by adding to the base a mound of earth termed a banquette. * * * While these levee sections have resulted from practical experience rather than from theoretical considerations, it may be noted that the sections most generally employed in recent years insure an angle of repose of dry earth on wet clay of 14° when water is at the top of the levee, the limiting angle of such materials."

From these suggestions by the writer was evolved the standard levee section of the Commission (Fig. 1) which has been used on a levee line more than 1 400 miles in length during a period of more than 30 years, and without a failure from weakness of section until the flood of 1927. During this flood, a disastrous crevasse occurred at Mounds, Miss., in a levee of standard section. A succession of rains converted the earth composing the parts of the levee above the banquette into a mud which could not withstand the static pressure due to a head of water of 6 ft., and it is natural to assume that the material placed in that part of the embankment was unsuited for levee construction.

When the writer was Assistant to the Commission and found material of such a character, he did not hesitate about enlarging the levee section which he had recommended. Such a course of action was so self-evident that he did not even consider it necessary to obtain the approval of the Commission for this change in form. It is illustrative of the hysteria that is sweeping over the country as a result of the flood that it is now proposed to prohibit the Assistant from exercising such a discretionary power, and to enlarge the

entire levee line to a section which will resist the water pressure, even if composed of poor material, notwithstanding the fact that more than 1 400 miles of levees of standard section, or weaker, successfully resisted the flood of 1927. This proposed reform will cost about $40 000 000.

Before making such an expenditure, however, it is advisable to make a further investigation of the causes of the conversion of this levee into mud. Several other levees, while not failing, showed similar signs of weakness, but it is a peculiarity of this flood alone. In all previous floods a mound of earth 6 ft. high, with an 8-ft. crown and slopes of 1 on 3, properly compacted and covered with a layer of Bermuda sod, could resist the water pressure no matter how porous the soil might be, of which it was composed, and if not protected by a revetment of Bermuda sod, rains eroded its slopes, instead of soaking into the mass and destroying its cohesion. During the flood of 1892, the writer, who was then Assistant to the Commission, had to hold a levee line 40 miles long in the Tensas Basin in Arkansas, a great part of which had a crown of 4 ft. and slopes of 1 on 2, not only until the water attained the top of the levees, but, by means of sand-bags and other devices, to still greater heights, and the only material that was converted into mud by rains was the fresh earth used in topping.

The Commission has passed strict regulations about allowing vehicles on the levee crown. If, however, during the excitement of flood protection, employees ignored those regulations and used automobiles for inspection purposes, the Bermuda sod would be rapidly destroyed during rainy weather, and longitudinal ruts formed from 6 in. to 1 ft. deep. With a 14-in. rainfall in 24 hours, these ruts would become veritable reservoirs for retaining water which could seep into the levee, and convert the best material into a mud. The excessive sloughing of the land slope of levees during this flood may also be due to this cause.

While the writer has no knowledge that such was the practice, before expending $40 000 000 on the levee line to enlarge the section, he would try the experiment of giving steep slopes to the crown, which would not only facilitate the run-off of the rainfall, but render it physically impossible for automobiles to violate the Commission's regulations. It is noted that the various projects for enlarging the levee section invite a greater use of the crown for travel in vehicles by widening it.

Conclusion

Congress has been extremely liberal in its appropriations for the Mississippi River since the creation of the Commission, but any one who assumes that Congress while limiting the expenditures for all the other rivers and harbors in the United States to $50 000 000 per year, can be persuaded to appropriate for the Mississippi alone $50 000 000 to $100 000 000 per year during a period of 10 years, has little knowledge of the workings of the Congressional mind.

Secretary Hoover has recently called attention to seven great projects of water development which will cost from $1 500 000 000 to $2 000 000 000, and there are numerous other expensive river and harbor projects which have

local support. The American people will most liberally contribute a vast sum to the Red Cross for humanitarian purposes, but when it is a question of expenditure from the United States Treasury, every community will demand consideration of its local needs. An equitable distribution of the funds over the entire country is necessary to insure the passage of a river and harbor bill.

The Mississippi River Commission has always been embarrassed by a lack of funds, and the fundamental principle governing its actions during the past 45 years has been to benefit the people of the delta as much as was possible with the appropriations made by Congress and under the limitations prescribed by that body. It is probable that it will have to pursue the same course in the future. If the attempt to establish a grade line which would be above all probable floods, as recommended by the writer when he was President of the Commission, should be abandoned, and the original Commission grade of 3 ft. above the stages that have occurred at the different stations along the river be substituted for it, there would be a probability of carrying out such a program with the funds which can be anticipated. Such a project not only would meet the economic situation, but would be in accordance with normal engineering practice.

The city engineer who designs sewers to carry off the rainfall has to solve the question, "What is the maximum run-off for which provision must be made?" The *Transactions* of the Society contains a number of discussions of this subject, and curves have been constructed showing graphically the probability of run-offs extending to periods of several hundred years. It is, however, generally recognized by the Engineering Profession that provision of a sewer system for a city which will prevent the flooding of the streets under the most adverse conditions, is an unwarranted extravagance. For example, if the City of New Orleans had constructed a sewer system to provide for the rainfall which occurred during the recent flood, it would have been compelled to expend on it, funds which could have been more profitably diverted to street pavements, to a water supply, and to other essentials to a city's progress. The occasional over-taxing of the sewer system is less injurious to the city's development than the curtailment of other prime necessities.

When the problem is expanded from drainage of a city to that of a continent, its complexity increases and the determination of future possibilities becomes more uncertain. Such possibilities are not then determined by mathematical computations, but by the imagination. When it is stated that a flood like that of 1927 will not probably occur again in 100 years, the engineer must take into consideration probabilities instead of possibilities, and consider the interest on investment for a period of 100 years. If the interest is computed at 4% on the cost of some of the proposed projects, it would be greater in 15 years than the estimated damage caused by the flood. A disaster which will not probably occur once in 100 years is in the same category as an earthquake or cyclone. It is an Act of God for which the philanthropist should make provision. Because an earthquake formed Reelfoot

Lake opposite New Madrid, Mo., 100 years ago, engineers do not make the buildings in the Mississippi Valley earthquake-proof.

It is suggested, however, that the engineer who attempts to solve the problem must make a definite decision whether he will be governed by probabilities or by possibilities. To submit a project which in one part provides only for what will probably occur, while in another makes provision for a possible increase, will be trying to serve two masters, with the usual result.

The final suggestion is also submitted that no matter how much money is expended on the levee line, it will still be at the mercy of the combination of a muskrat, a dark night, and a careless levee inspector. A burrowing animal can create a hole in the finest levee that has been devised, which, if not closed within a few moments, will insure its destruction. Perfection is difficult in all engineering enterprises. To try to construct a levee line which will resist the river flow under all the imaginary conditions which now fill the public mind, is to attempt the impossible.

RAINFALL CHARACTERISTICS OF THE MISSISSIPPI DRAINAGE BASIN

By H. C. Frankenfield,* Esq.

The Mississippi Basin

The drainage basin of the Mississippi River comprises about all that portion of the United States lying between the Allegheny and the Rocky Mountains except the Great Lakes and Hudson Bay drainages, with a total area of 1 250 900 sq. miles, about two-fifths of the total area of the United States proper. Some water from 31 of the 48 States passes into the Gulf of Mexico. For convenience of study it has become customary to divide the entire area into six major sub-divisions, or sub-basins, as follows: the Missouri, the Upper Mississippi, the Ohio, the Arkansas-White, the Red, and the Lower Mississippi. The drainage areas of the sub-basins, together with their percentages of the entire area are as given in Table 1.

TABLE 1.—Drainage Area of the Mississippi Basin.

Sub-basin.	Drainage area, in square miles.	Percentage.
Missouri	528 850	42.3
Upper Mississippi	187 850	15.0
Ohio	203 900	16.3
Arkansas-White	186 000	14.9
Red	90 000	7.2
Lower Mississippi	54 300	4.3
Total	1 250 900	100.0

Barometric Pressure Distribution and Precipitation

Having these proportions in mind in a general way, consider the precipitation that occurs over each of these sub-basins, its form to a certain extent, the peculiarities of its distribution, and the reasons therefor, so far as one may be able to give them. Without going deeply into the subject, begin with the ordinary weather chart. Certain relative distributions of barometric pressure will be followed by fair weather, and others by rain or snow. These different distributions have long been grouped into types that officials of the U. S. Weather Bureau have classified according to their regions of inception so far as they have been able to determine them. The types that are principally concerned in precipitation over the Mississippi Drainage Basin are known as the North Pacific, the South Pacific, the Central Rocky Mountain, and the Southwestern. Reference is had now to low-pressure types only, or, technically, cyclones, as these are directly concerned in precipitation causation, although influenced in greater or less degree by the high-pressure types, or anti-cyclones.

Low-pressure areas have warm and moist southeast and south winds on their eastern and southern sides, and the warm and moist masses of air rise until they come into contact with other masses of air which are sufficiently cold to cause condensation of some of the water vapor in the warm masses, and rain

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or snow results. The amount of resulting precipitation will depend upon the velocity and direction of movement of the cyclone and attendant anti-cyclone, the amount of available water vapor, and the difference in temperature between the warm cyclone and the cold anti-cyclone.

The great general and never failing type of rain producer, whatever its origin may be, is a well-defined low-pressure area moving in some easterly direction—usually northeast or east-northeast—with a relatively cold high-pressure area to the northward or northeastward. This is true for the entire Mississippi River Basin, notwithstanding the varying amounts of precipitation that are normal to the different sub-basins. As a matter of fact the four types previously mentioned are neither more nor less than modifications of the great general type, but with different points of origin and different velocities of progression.

Normal Precipitation Over the Mississippi Basin

The figures in Table 2 show the normal precipitation by seasons for the States comprised in the Mississippi Basin. The data are given for the States in each of the six major sub-basins, and wherever a State extends over portions of adjacent sub-basins, the figures cover only that portion of the State within that particular sub-basin. For instance, Kansas would appear in both the Missouri and Arkansas Sub-Basins, etc.

TABLE 2.—Seasonal Normal Precipitation by States, Mississippi Basin.

State.	Drainage area, in square miles.	Precipitation, in Inches.				
		September-November.	December-February.	March-May.	June-August.	Total.
Missouri Sub-Basin.						
Montana	121 588	3.00	1.81	4.40	5.75	14.96
Wyoming	69 908	3.15	2.34	4.88	4.24	14.61
Colorado	29 457	3.13	2.02	5.67	6.18	17.00
North Dakota	41 083	2.79	1.48	4.71	7.76	16.74
South Dakota	76 609	3.80	1.79	6.13	8.51	20.23
Minnesota	2 022	5.22	1.49	6.05	10.52	23.28
Nebraska	77 520	4.91	2.09	7.44	10.55	24.99
Iowa	17 289	7.31	3.02	9.13	11.83	31.29
Kansas	42 199	5.96	2.63	7.98	10.57	27.14
Missouri	36 009	8.82	5.17	11.32	12.51	37.82
Canada	15 216	....	....	.....	.....	.....
Entire area	528 850	4.81	2.39	6.77	8.84	22.81
Arkansas-White Sub-Basin.						
Colorado	29 303	3.08	2.51	5.16	6.81	17.56
Kansas	39 959	6.21	3.03	8.28	10.38	27.90
Missouri	14 134	9.68	6.96	13.08	12.81	42.53
New Mexico	17 229	3.49	1.51	4.15	7.65	16.80
Oklahoma	44 057	7.77	4.16	10.36	9.92	32.21
Arkansas	28 013	10.17	10.72	14.52	11.66	47.07
Texas	13 305	4.88	2.15	6.06	7.97	21.06
Entire area	186 000	6.47	4.43	8.80	9.60	29.30

TABLE 2.—(*Continued.*)

State.	Drainage area, in square miles.	Precipitation, in Inches.				
		September-November.	December-February.	March-May.	June-August.	Total.
Upper Mississippi Sub-Basin.						
South Dakota	1 006	4.57	2.43	7.04	9.54	23.58
Minnesota	48 296	5.94	2.28	6.83	11.16	26.21
Wisconsin	38 187	8.10	3.37	8.18	11.61	31.26
Iowa	38 908	7.74	3.59	9.46	11.80	32.59
Illinois	45 448	8.38	6.16	10.48	10.56	35.58
Indiana	2 369	8.34	6.59	10.36	10.21	35.50
Michigan	273	....	....	.....	.....	.....
Missouri	13 363	8.95	6.66	11.32	11.07	38.00
Entire area	187 850	7.43	4.44	9.10	10.85	31.82
Ohio Sub-Basin.						
Illinois	11 413	8.68	8.12	11.36	10.60	38.76
Indiana	30 061	8.93	8.73	11.49	11.00	40.15
Ohio	30 250	8.17	8.86	10.71	11.62	39.36
Pennsylvania	14 680	8.57	9.46	10.50	12.61	41.14
New York	2 136	....	.....	.....	.....	.....
Maryland	300	8.43	10.99	11.82	13.17	44.41
Virginia	6 902	8.40	10.63	11.26	13.38	43.67
West Virginia	19 567	8.64	11.01	11.75	13.38	44.78
North Carolina	6 581	9.65	13.03	13.60	15.91	52.19
Kentucky	39 398	9.08	11.94	12.38	12.02	45.42
Tennessee	34 124	9.25	13.56	13.96	13.50	50.27
Alabama	6 926	9.16	14.55	14.68	13.15	51.54
Georgia	1 562	....	.....	.....	.....	.....
Entire area	203 900	8.81	10.99	12.14	12.76	44.70
Red Sub-Basin.						
Oklahoma	26 000	7.82	3.97	9.75	9.23	30.77
Arkansas	19 584	9.30	11.37	15.23	10.92	46.82
Texas	28 000	7.47	5.73	10.20	9.32	32.72
Louisiana	16 416	9.28	13.98	14.38	12.07	49.71
Entire area	90 000	8.47	8.76	12.39	10.38	40.00
Lower Mississippi Sub-Basin.						
Missouri	5,914	9.48	10.99	13.12	12.10	45.69
Arkansas	5.738	10.10	12.88	15.31	11.59	49.88
Louisiana	11.300	10.59	14.09	12.93	17.07	54.68
Kentucky	1.200	10.22	13.09	13.40	11.39	48.10
Tennessee	7.898	9.77	13.63	13.92	11.17	48.49
Mississippi	22.250	8.96	15.29	15.93	12.22	52.40
Entire area	54 300	9.85	13.33	14.10	12.59	49.87

Table 2 has virtually been brought down to date, an always desirable condition for those who are interested in the study of normal precipitation, and, furthermore, so far as the writer is aware, the data have not previously been presented in similar form, the usual practice having been to confine the com-

ponent data within the exclusive limits of State boundaries for which river drainage basins have no respect whatever. Table 3 gives the data in Table 2 arranged to show the normal annual precipitation for the sub-basins weighted in accordance with the ratios between their respective drainage areas and the area of the entire basin.

TABLE 3.—Weighted Normal Precipitation, Mississippi Basin.

Sub-basin.	Drainage area, in square miles.	Normal precipitation in inches.	Percentage.
Missouri	528 850	9.62	30.9
Upper Mississippi	187 850	4.78	15.4
Ohio	203 900	7.29	23.4
Arkansas-White	186 000	4.36	14.0
Red	90 000	2.88	9.3
Lower Mississippi	54 300	2.17	7.0
Total	1 250 900	31.10	100.0

It is readily apparent from even a casual inspection of Tables 2 and 3 that the precipitation over the Mississippi Drainage Basin is at a minimum at its extreme northern and western boundaries, and increases steadily, although by no means uniformly, to the eastward, southeastward, and southward. The meteorological reasons for this distribution are not intricate, and are undoubtedly familiar to the majority of engineers.

Barometric Pressure and Geographical Distribution of Precipitation

As previously stated, low-pressure areas, or cyclones, or storms, as the U. S. Weather Bureau terms them, if they have assumed definite and pronounced formation, as they move eastward over the country, are attended by warm southeast winds on their southeastern and southern sides, or in their southeastern quadrants. These masses of warm air, coming as they do from the Atlantic Ocean, or the Gulf of Mexico, carry with them large quantities of water vapor, and as the air rises higher and higher in the atmosphere, it finally becomes cooled through loss of heat by expansion and by contact with cooler air masses. The ascending air, as it cools, of course loses its capacity for retaining vapor, the saturation point is soon reached, and condensation, or precipitation, sets in. The land nearest the original water supply naturally receives the largest quantity of precipitation, and as the air continues to move farther inland toward the northwest, the available supply of moisture becomes less and less, until the water supply available at the existing temperature has been precipitated, generally at a considerable distance from the Northwestern Rocky Mountains. This is the usual occurrence, although not without exception so far as the surface weather chart shows. At times, there are heavy rains, occasionally torrential, that attend the summer thunderstorms between the Missouri River and the Rocky Mountains. The surface pressure distribution preceding these heavy rains does not always present an orderly arrangement, but aerological observations by kite and balloon will generally show a

mass of cold air, unusually cold for the season, to the westward and northwestward, close to the warm air masses, which will soon under-run them with resulting heavy precipitation.

The Far Northwest then does not receive much moisture from the Atlantic Ocean and the Gulf of Mexico, as the available supply is exhausted farther to the eastward and southward, and its precipitation is due to other types of pressure distribution. One of these is the presence of a warm low-pressure area over Southeastern Wyoming, or Eastern Colorado, with a cold high-pressure area over Saskatchewan, and precipitation, either of rain or snow, will result from the cold northerly and northeasterly winds. This precipitation is usually heavy. Aside from this type precipitation over the Far Northwest generally results from the cold air of the high areas following the movement of low areas across that section. Over the great central valleys the precipitation precedes and accompanies the low area, with little or none following, but in the Far Northwest the southeasterly and southerly winds blowing into the eastward-moving low areas carry but little moisture, not enough to condense until the cold from the following high area reaches it, and precipitation will set in behind the low area and with west and northwest winds. Precipitation from this type of pressure distribution is uniformly small.

Finally, the one great rain-producing type for the entire Central and Southern Mississippi Basin is the "Southwestern." By "Southwestern" is meant a low-pressure area from Mexico, Arizona, or New Mexico (sometimes originating in the Central Pacific Ocean), moving northeastward through Texas and the Ohio Valley, with the usual high area to the northward. The moisture supply is virtually unlimited, and the precipitation is abundant, sometimes in the form of heavy snow in the Ohio and Middle Mississippi Valleys. These storms afford a sufficient explanation of the heavier precipitation over the central and southern portions of the basins and incidentally a partial reason at least for flood behavior in the Lower Ohio and Lower Mississippi Rivers. The rains begin first over these sections, and by the time they reach the Upper Ohio Valley the rivers below have risen decidedly, so that the waters from the great and prolific tributaries of the Ohio and from the upper river itself are added to the already swollen streams below. Sometimes these southwestern storms move in series with intervals of 3 or 4 days, and when this happens a severe flood follows. In 1912 there was a series of six of these storms, separated by intervals of only a few days, and the resulting flood was one of the greatest in the history of the Lower Ohio and Lower Mississippi Valleys.

Seasonal Distribution of Precipitation

A study of the seasonal distribution of precipitation over the Mississippi Basin discloses a number of points of difference when the sub-basins are examined separately. In the Missouri and Upper Mississippi Basins the minimum amount occurs during the winter months, December to February. Nearly all the precipitation is in the form of snow which produces comparatively little water when melted. There is a decided increase during the spring

months, March to May, and a still further but less decided increase during the summer months, June to August, a large part of the summer rains coming from thunderstorms. During the autumn months, September to November, there is a sharp decrease as the thunderstorms become less frequent and storm activity is usually at its lowest ebb. In the Ohio Basin there is a steady increase from season to season, beginning with the autumn, although in Illinois and Indiana the winter precipitation is lower on account of the snow effect. The spring and summer data do not differ greatly, the summer thunder showers supplying slightly more rain than the spring storms. In the Arkansas-White Basin the tendency is the same as in the Missouri and Upper Mississippi Basins. The winter decrease from snow effect is not so marked, as it is mostly limited to the western portion of the drainage area, and, furthermore, the rains over the States of Missouri and Arkansas are abundant. In the Red and Lower Mississippi Basins the minimum amount of precipitation is recorded during the autumn months, and the maximum during the spring months, the season of the most effective rain-producing storms. The summer months have less rain than the spring months as summer pressure conditions are usually more or less stagnant, and thunderstorms, while frequent, are often local and spring rains are general. Table 4 shows the seasonal amounts for each sub-basin and the percentage of the total fall.

TABLE 4.—Normal Seasonal Precipitation, in Inches, and Percentages of Normal Annual Amounts, Mississippi Basin.

Sub-basin.	September-November.		December-February.		March-May.		June-August.		Total.	
	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.
Missouri	4.81	21	2.39	11	6.77	29	8.84	39	22.81	100
Upper Mississippi	7.43	23	4.44	14	9.10	29	10.85	34	31.82	100
Ohio	8.81	20	10.99	25	12.14	27	12.76	28	44.70	100
Arkansas-White	6.47	22	4.43	15	8.80	30	9.60	33	29.30	100
Red	8.47	21	8.76	22	12.39	31	10.38	26	40.00	100
Lower Mississippi	9.85	20	13.33	27	14.10	28	12.59	25	49.87	100
Total weighted for areas	6.59	21	5.32	17	9.02	29	10.17	33	31.10	100

Snowfall

The correct measurement of snowfall has never proved to be an easy problem. It is true that improved apparatus and methods have resulted in very much greater accuracy during the last thirty years so far as individual places of observation only are involved, but when attempts were made to co-ordinate these individual data over areas of any considerable extent, difficulties arose. Sometimes it was found that the total measured run-off of a stream was greater than the total recorded amount of precipitation of rain

and snow combined, while at other times the percentage of run-off was less than could be reasonably expected. In both instances the trouble was due much more to deficient data than to defective methods of observation. Over broad expanses of comparatively level country this objection could sometimes be waived, but in mountainous regions and in others of diversified topography it is ever-present.

For many years it was impossible, except in a few isolated instances, to obtain snowfall data for the high mountain altitudes, although in later years the U. S. Weather Bureau, ably assisted by the U. S. Forest Service, has been able to reach some of the high places. Nevertheless, there remains the impossibility of accomplishing satisfactory co-ordination of the data on account of the limited areas covered and the numerous and abrupt changes in topography. If the work of snowfall measurement and its translation into water equivalent is to be properly performed there must be made first a topographical survey of the mountain areas and perhaps some others. Once the survey is made it will stand for all time. Then must come the location of a number of measuring stations that will be sufficiently representative of the entire area; that is, located where they will afford a true index of the snow depths, and entirely without any idea as to uniformity with respect to relative location. The imperative need is for more data properly obtained. During recent years several attempts have been made to secure funds from Congress for a few experimental surveys, but without success. However, there are hopes of the future, especially in view of the fact that the great floods of 1927 appear to have nationalized the water problem, at least to a very large extent. The results of a computation of the normal annual snowfall by the ordinary method of averages are given in Table 5, for the States of the Mississippi Valley, with the small areas of North Carolina, Maryland, Western New York, Michigan, Texas, and New Mexico omitted.

TABLE 5.—NORMAL ANNUAL SNOWFALL, MISSISSIPPI BASIN.

State.	Normal annual snowfall, in inches.	State.	Normal annual snowfall, in inches.	State.	Normal annual snowfall, in inches.
Eastern and Central		Kansas	17	Kentucky	16
Montana	55	Oklahoma	8	Tennessee	10
Eastern Wyoming	59	Wisconsin	45	Northwest Georgia	4
Eastern Colorado	72	Illinois	23	Northwest Alabama	4
North Dakota	82	Indiana	26	Mississippi	2
South Dakota	36	Ohio	32	Arkansas	8
Nebraska	30	Western Pennsyl-		Louisiana	1
Minnesota	39	vania	47	....	....
Iowa	30	West Virginia	34	....	....
Missouri	20	Virginia	18	....	....

As would be expected, the greatest snowfall occurs over the Rocky Mountain districts, with the maximum of about 72 in. in Eastern Colorado. From the mountains eastward and north of the 35th Parallel there is a decrease to the Middle Mississippi Valley, except in Minnesota and Wisconsin, which are partly under the influence of the Great Lakes. Then follows an increase until

the Allegheny Mountains are reached, with a maximum of about 47 in. over Western Pennsylvania. Snowfall below the 35th Parallel is not of consequence.

The assemblage of the figures in Table 5 into normals for the individual sub-basins could not be accomplished without more time than was available, but roughly they are as follows:

Basin.	Depth, in inches.
Missouri	39
Upper Mississippi	31
Ohio	26
Arkansas-White	13
Red	6
Lower Mississippi	5

On the usually assumed arbitrary basis of 1 in. of water for each 10 in. of snow, the precipitation of the entire Mississippi Basin, using the data in Table 2, may be divided as shown in Table 6.

TABLE 6.—Normal Rain and Snow, Mississippi Basin.

Basin.	Rainfall, in inches.	Snowfall (melted), in inches.	Total, in inches.
Missouri	18.91	3.90	22.81
Upper Mississippi	28.72	3.10	31.82
Ohio	42.10	2.60	44.70
Arkansas-White	28.00	1.30	29.30
Red	39.40	0.60	40.00
Lower Mississippi	49.37	0.50	49.87
Entire basin	28.31	2.79	31.10
Weighted	(91%)	(9%)	(100%)

Methods of Computing Normal Precipitation

The normal values in Table 2 were computed in accordance with the time-honored procedure; that is, by dividing the sum of the normal values at as many units of observation as possible by the total number of units. Little objection could be offered to this plan were it not for the fact that, except in rare instances, the aggregate quantity of data for any considerable area is not sufficient to insure a true and dependable normal. Every hydraulic engineer knows that the amount of precipitation that occurs over so small an area as several hundred square miles is far from uniform, even under virtually identical conditions of topography and meteorological influence. In other words, under the system that has universally prevailed—a system that economic conditions, if nothing more, has rendered compulsory in this country—the data have always been insufficient to serve in the most efficient manner the interests for which they were recorded.

If the number of stations of observation could be multiplied by three or four, or if the stations could be located on the basis of one for not more than each 50 or 75 sq. miles of area, a solution of the problem of questionable normals would be obtained. In mountainous sections, especially in high moun-

tains with their abruptly diversified topography, the stations of observation should be more numerous than one for each 50 or 75 sq. miles, and arranged so as to have proper regard for rapidly changing topography. The Weather Bureau has long realized this situation, but thus far it has been helpless through inability to finance the undertaking. No criticism of other Governmental agencies is implied in this statement, as the Weather Bureau has probably received all the consideration that has been due and in keeping with the requirements of other services of equal importance. Therefore, with a view to the better utilization of the available data in connection with the discussion of flood problems, the Weather Bureau began five years ago to compile precipitation data by another method which, if laborious, at least possesses the merit of being more accurate, so that the results obtained were very probably as nearly correct as they would have been had the number of stations of observation been multiplied by three or four.

If a planimeter is not available, the plan suggested to the writer by Professor C. F. Marvin, Chief of the Weather Bureau, may be used. It is as follows: Monthly data for a large number of stations were charted and isohyetal lines carefully drawn. These lines were then traced upon sheets of cross-section paper, together with the outlines of the six drainage areas.

The isohyets divide the drainage basins into various small sub-areas, over which the precipitation may be assumed to be uniform and of an amount represented by the mean between the two adjacent isohyetals. Therefore, the number of squares in each sub-area was counted. This number was then multiplied by the average precipitation for the sub-area in question and the product divided by the sum of the counts for all the sub-areas, which latter, of course, is the number of squares in the whole drainage basin being studied. Finally, the sum of the quotients found in the manner described, gives the depth of precipitation which, spread uniformly over the whole basin, would represent the same quantity of water as fell in the irregularly distributed precipitation.

This method of discussing precipitation permits the consideration of actual quantities of water over large areas with the certainty that inequalities of rainfall distribution and the uncertainties arising therefrom have been largely eliminated. The method was used in the computation of precipitation data for the great Mississippi floods of 1922 and 1927, and for purposes of comparison the data for the floods of 1882, 1912, and 1913 were computed on the same basis. Monthly, seasonal, and annual normals for the sub-basins and for the different States were also computed, and the sub-basin data are given in Tables 7 and 8 herewith.

Analysis of Distribution of Precipitation

General Distribution.—Table 7 shows that the maximum water-fall over the Mississippi Basin occurs in May and June and in equal amount for each month; also, that the minimum fall occurs from November to February, with a maximum difference of only 0.15 in. between any two months.

Considering the sub-basins individually the same general arrangement is found in the Upper Mississippi, the Missouri, and the Arkansas-White Basins,

those that have the greatest amount of snowfall. The distribution in the Ohio Basin is different in that the maximum fall occurs in March and July and the minimum during October and November, instead of in the winter. In the Red Basin the maximum fall occurs in April and May and the minimum in February, while in the Lower Mississippi Basin the maximum occurs in March, April, and December, and the minimum in October, although, as a whole, there is little variation in monthly values, except during the autumn months.

TABLE 7.—WEIGHTED MONTHLY MEANS OF PRECIPITATION OF THE INDIVIDUAL BASINS OF THE MISSISSIPPI DRAINAGE AREA.

(Precipitation, in Inches.)

Basin.	Percentage of area.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Annual.
Upper Mississippi...	15.0	0.21	0.20	0.30	0.42	0.62	0.63	0.54	0.51	0.51	0.36	0.26	0.20	4.76
Missouri ...	42.3	0.30	0.33	0.49	0.85	1.29	1.36	1.08	0.92	0.84	0.60	0.35	0.32	8.73
Ohio	16.3	0.64	0.53	0.70	0.62	0.64	0.69	0.70	0.63	0.50	0.47	0.47	0.59	7.18
Arkansas-White....	14.9	0.21	0.22	0.32	0.46	0.57	0.50	0.47	0.47	0.39	0.33	0.26	0.24	4.44
Red.........	7.2	0.18	0.17	0.23	0.31	0.32	0.27	0.23	0.23	0.20	0.21	0.19	0.22	2.76
Lower Mississippi...	4.3	0.21	0.19	0.22	0.22	0.19	0.18	0.20	0.18	0.14	0.12	0.15	0.22	2.22
Total.....	100.0	1.75	1.64	2.26	2.88	3.63	3.63	3.22	2.94	2.58	2.09	1.68	1.79	30.09

TABLE 8.—WEIGHTED SEASONAL PRECIPITATION OF THE INDIVIDUAL BASINS OF THE MISSISSIPPI DRAINAGE AREA.

(Precipitation, in Inches.)

Sub-basin.	Ratio to whole basin.	FALL.	WINTER.	SPRING.	SUMMER.	Annual.
		September to November.	December to February.	March to May.	June to August.	
Upper Mississippi.............	0.150	1.13	0.61	1.34	1.68	4.76
Missouri..........................	0.423	1.79	0.95	2.63	3.36	8.73
Ohio................................	0.163	1.44	1.76	1.96	2.02	7.18
Arkansas-White................	0.149	0.98	0.67	1.35	1.44	4.44
Red..................................	0.072	0.60	0.57	0.86	0.73	2.76
Lower Mississippi............	0.043	0.41	0.62	0.63	0.56	2.22
Total	1.000	6.35	5.18	8.77	9.79	30.09

It will be noticed that the Upper Mississippi and Arkansas-White Sub-Basins differ but little as to drainage area and water depth, whereas the Missouri Basin, with nearly three times their drainage area, has less than twice the quantity of water on account of the long snow season. The Ohio Basin, with a drainage of a little less than two-fifths that of the Missouri Basin, has

82% as much water, the winter season showing an excess over the Missouri Basin. The Red Basin, with about one-half the drainage area of the Arkansas-White and Upper Mississippi, has about 62% and 58%, respectively, of the quantity of water of the two latter basins, while the Lower Mississippi Basin, with about 10% of the drainage area of the Missouri Basin, has about one-fourth as much water, but the Ohio Basin, with a drainage area about two and one-fourth times as large as that of the Red, has nearly three and one-fourth times as much water on account of the smaller precipitation over the Upper Red Basin.

Seasonal Distribution.—The seasonal distribution will conform, of course, closely in character to that previously outlined in the discussion of actual precipitation. A comparison of the annual normals and their percentages of the total basin precipitation by both methods of computation is given in Table 9.

TABLE 9.—Comparison of Precipitation Normals.

Sub-basin.	Actual Precipitation from State Normals.		Computed Precipitation.	
	Inches.	Percentage.	Inches.	Percentage
Missouri	9.62	30.9	8.73	29.0
Upper Mississippi	4.78	15.4	4.76	15.8
Ohio	7.29	23.4	7.18	23.9
Arkansas-White	4.36	14.0	4.44	14.7
Red	2.88	9.3	2.76	9.2
Lower Mississippi	2.17	7.0	2.22	7.4
Total	31.10	100.0	30.09	100.0

The small difference between the two totals is due to the fact that the State normals, after their reduction to the sub-basin normals, were then weighted according to the area ratios. Had the actual precipitation figures been used throughout, the difference would doubtless have been greater, but it was not feasible to undertake the work at this time, and the data were not obtainable from any other source.

Floods

So much for the causes, character, and distribution of precipitation over the great Mississippi Basin. It is in order now to discuss what becomes of this precipitation when at times its normal characteristics undergo decided changes, that is, become abnormal as to quantity, time, and distribution, and manifest themselves in the form of the floods that have become so distressingly and destructively frequent during the last 16 years, and especially during 1927. In keeping with the subject assigned to the writer, floods will not be discussed except in connection with the precipitation that caused them, other flood problems being beyond his province.

Before taking up the Lower Mississippi floods, the floods of the Missouri and Upper Mississippi Basins will be touched upon briefly.

Severe or even ordinary floods in the Missouri River above the mouth of the Kansas River are rare. The greatest general flood of which there is record occurred in the spring of 1881 and extended to the mouth of the river, merging there with an Upper Mississippi flood of fair proportions. However, it was not very severe east of Kansas City, Mo. There was also a severe flood in March, 1887, in the North Dakota portion of the Missouri River, but the flood stage was not reached as far south as Omaha, Nebr.

These Upper Missouri floods are spring floods, and are due usually to a combination of melting snows in upper reaches, high temperatures, and rain. The melted snow factor is not of importance unless in large quantity, and supplemented by substantial rains, and the rain is the essential factor. It is not necessary to discuss the floods caused by the formation and subsequent breaking of ice gorges, and, incidentally, it is very probable that the flood of 1881 was largely an ice flood. From Kansas City eastward, Missouri River floods are quite frequent, although as a rule not great. They are essentially rain floods and occur usually in late June and in July. By far the greatest of these floods occurred in June, 1844, a season of torrential rainfall that also extended over the Middle Mississippi Valley causing stages that have not since been reached between the mouth of the Illinois River and Cape Girardeau, Mo.

Upper Mississippi Floods, that is, floods above the mouth of the Missouri River, are more numerous. They are almost entirely spring floods, and, unless caused by ice gorges, they are rain floods, although the rain may be considerably augmented at times by melted snows. Owing to the flat contour, these floods are likely to be destructive below the mouth of the Wisconsin River.

The floods of the Ohio, Arkansas-White, Red, and Lower Mississippi Valleys are virtually of annual occurrence, and great floods are not infrequent, as witness those of comparatively recent occurrence, 1912, 1913, 1915, 1916, 1920, 1922, and 1927. These floods are pre-eminently rain floods, although at times considerable assistance is lent by melting snows in the winter or early spring from the Ohio River and its many large mountain tributaries.

The Flood of 1927

As the flood of 1927 overshadowed all others from Cairo, Ill., southward it will hardly be necessary to discuss the precipitation conditions for any others except the flood of 1922 which was next in magnitude to that of 1927. All data will be expressed in terms of water depth over the entire area.

In Table 10 will be found the data for the floods of 1922 and 1927, together with the departures from the normals. The normals are of recent determination and are the product of the combined labors of Messrs. Montrose W. Hayes and Walter J. Moxom, of the U. S. Weather Bureau Office, at St. Louis, Mo., and engineers of the office of the Mississippi River Commission, also of St. Louis.

The total depth of water more or less contributory to the flood of 1927 was 10.79 in., and 12.38 in. if the last two weeks of December, 1926, are included, as they really should be. The excess of 1927 over 1922 for the entire period, December 18 to April 30, was 1.80 in., but for the four months, January to April, 1927, only 0.21 in., or a little less than 2 per cent. The excesses

above the normal amounts in both years are significant. The water depths for January were below the normal amounts in both 1927 and 1922, and February, 1927, was also deficient, while February, 1922, shows an excess. Taken together, January and February, 1927, show a deficiency of 0.22 in. against a deficiency of 0.15 in. in 1922, a difference so small that during the two months both years were virtually on even terms. The month of March shows a decided excess for both years, but with 1922 still in the ascendancy, the figures being + 0.72 in. for 1927 and + 1.24 in. for 1922. Both values are quite large and indicate a severe flood. In April, the month of greatest water depth over the entire area, conditions became decidedly reversed with an excess of 1.75 in. in 1927 against 0.94 in. in 1922.

TABLE 10.—PRECIPITATION FOR THE FLOODS OF 1922 AND 1927, IN TERMS OF INCHES OF WATER OVER THE ENTIRE MISSISSIPPI DRAINAGE AREA, AND DEPARTURES FROM NORMAL.

(Absence of sign indicates plus departure.)

Sub-area.	Drainage, in square miles.	Flood of 1922.									
		January.		February.		March.		April.		Total.	
		Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.
Upper Mississippi......	187 850	0.16	—0.05	0.31	0.11	0.35	0.05	0.53	0.11	1.35	0.22
Missouri..........	528 850	0.29	—0.01	0.44	0.11	0.89	0.39	1.48	0.63	3.10	1.12
Ohio..........	203 900	0.44	—0.21	0.41	—0.12	0.92	0.21	0.66	0.04	2.43	—0.08
Arkansas-White........	186 000	0.18	—0.03	0.21	—0.01	0.60	0.28	0.62	0.17	1.61	0.41
Red..........	90 000	0.19	0.01	0.22	0.05	0.40	0.17	0.36	0.05	1.17	0.28
Lower Mississippi......	54 300	0.19	—0.02	0.21	0.02	0.36	0.14	0.16	—0.06	0.92	0.08
Total..........	1 250 900	1.45	—0.31	1.80	0.16	3.52	1.24	3.81	0.94	10.58	2.03

Sub-area.	Flood of 1927.										December 18–31, 1926.		1927 total, including December 18–31, 1926.	
	January.		February.		March.		April.		Total.					
	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.
Upper Mississippi......	0.15	—0.06	0.15	—0.05	0.40	0.10	0.64	0.22	1.34	0.21	0.08	—0.01	1.42	0.20
Missouri..........	0.25	—0.05	0.28	—0.05	0.63	0.14	1.59	0.74	2.75	0.78	0.07	—0.07	2.82	0.71
Ohio..........	0.67	0.02	0.54	0.01	0.82	0.11	0.95	0.33	2.98	0.47	0.70	0.43	3.68	0.90
Arkansas-White........	0.30	0.09	0.16	—0.06	0·48	0.16	0.69	0.24	1.63	0.43	0.22	—0.11	1.85	0.32
Red..........	0.17	—0.01	0.17	0.00	0·29	0.06	0.42	0.11	1.05	0.16	0.25	—0.15	1.30	0.01
Lower Mississippi......	0.16	—0.05	0.18	—0.01	0.37	0.15	0.33	0.11	1.04	0.20	0.27	—0.17	1.31	0.03
Total..........	1.70	—0.06	1.48	—0.16	2.99	0.72	4.62	1.75	10.79	2.25	1.59	—0.08	12.38	2.17

Comparison of the data as a whole for 1922 and 1927 does not disclose any particular reason why the magnitude of the flood of 1927 should have exceeded

that of 1922. In fact, the behavior was much the same in both. The months of January and February differed but little, and, taken together, both were deficient. In March, 1922, the excess precipitation was large and the maximum flood occurred in that month, while in April, 1927, the excess was nearly twice as great as in April, 1922, and the maximum and greater flood occurred in that month. Therefore, it is obvious that the distribution of precipitation, as to intensity, time, and locality, is almost as important as the amount thereof. In both 1922 and 1927 a reasonable additional amount in January and February and an equivalent lesser amount in March and April would have lowered the flood crests materially, while the reverse occurrence would have resulted in still higher crests, assuming that levees had remained intact. In 1912, another year of great March and April flood, the excess water in February and March amounted to 1.57 in., with a slight deficiency in January, and the total quantity of water for the three months was 8.24 in., 2.34 in. less than in 1922, and 2.55 in. less than in 1927. The 1912 flood was caused by a series of six storms of the "southwestern" type, the best rain producers, as stated previously, for the Middle and Lower Mississippi Basins. These six storms were separated by intervals of only a few days, a period entirely too short to check the rate of rise materially, and another inch or so of water from the storms would probably have resulted in a flood as great as that of 1922 had the levees remained intact. It should be stated, also, that during the flood of 1912 neither the Upper Mississippi, Arkansas, nor Red Rivers was in severe flood.

In the floods of 1882, 1903, and 1913, the total quantities of water, January to March, inclusive, were 7.68, 6.11, and 6.44 in., respectively. These figures appear to justify the assumption, partly at least, that a water cover over the Mississippi Basin of about 10 in. in 3 or 4 months between January and April, will probably result in a great flood from Cairo southward, and that under the most favorable conditions for flood formation, a somewhat less amount will answer.

All data thus far given have been based upon precipitation and water depth over the entire Mississippi Basin, although it is a recognized fact that the Missouri River above the mouth of the Platte, and the Mississippi River above the mouth of the Wisconsin do not contribute materially to Lower Mississippi floods. If these upper areas, amounting to approximately one-third of the total drainage, were to be excluded, the precipitation over the remaining two-thirds would have amounted to 16.28 in. of water over the latter area in 1927, whereas in 1922 the total water depth was 13.70 in., 2.58 in. less than in 1927. If the data for the last two weeks in December, 1921, and 1926, were included, the 1927 excess would be still larger.

Possible Maximum Flood Crests in 1927

A question frequently asked the U. S. Engineer Corps and the Weather Bureau was this: What would have been the maximum crests reached during the flood of 1927 had all the main levees remained unbroken? Each organization considered the question independently and by somewhat different methods, and opportunity was afforded to make informal comparison of the Weather Bureau figures with those of the Engineer Corps in Washington.

It is gratifying to be able to state that differences in the two groups of figures were insignificant. The stages reached at key stations and the estimated possible stages under most favorable conditions are given in Table 11.

TABLE 11.—ACTUAL AND POSSIBLE CREST STAGES, FLOOD OF 1927.

Station.	River.	Flood stage, in feet.	Crest, 1927, in feet.	Possible stage, 1927, in feet.	Difference, in feet.
St. Louis, Mo.	Mississippi	30	36.1	36.1	0.00
Paducah, Ky	Ohio	43	47.2	48.0	+0.08
Cairo, Ill.	Ohio	45	56.4	57.7-58.0	+1.3-+1.6
Memphis, Tenn.	Mississippi	35	46.0	47.2-47.5	+1.2-+1.5
Helena, Ark.	Mississippi	44	56.8	58.2-58.5	+1.4-+1.7
Arkansas City, Ark.	Mississippi	48	60.5	68.5-69.0	+8.0-+8.5
Greenville, Miss.	Mississippi	42	54.7	61.5-62.0	+6.8-+7.3
Vicksburg, Miss.	Mississippi	45	58.7	64.5-65.0	+5.8-+6.3
Natchez, Miss.	Mississippi	46	56.5	64.5-65.0	+8.0-+8.5
Baton Rouge, La.	Mississippi	35	47.8	54.5-55.0	+6.7-+7.2
New Orleans, La.	Mississippi	17	21.0	27.2-27.7	+6.2-+6.7

The processes of reasoning through which the figures in Table 11 were evolved were basically alike, and perhaps a single example will suffice to illustrate them.

At Cairo the crest stage of 56.4 ft. occurred on April 20. The crevasse at Dorena, Mo., on the right bank, 30 miles below Cairo, occurred at 4:00 A. M., April 16, and after that time the river at Cairo rose only 0.7 ft., notwithstanding the fact that the Mississippi River at St. Louis was at a stage of 34 ft., or 4 ft. above the flood stage, and continued to rise steadily for a week thereafter. The Ohio River, at Paducah, Ky., also continued to rise for a few days after the crevasse. The rises at St. Louis and Paducah after the Dorena crevasse were about 2.0 and 0.9 ft., respectively, with an increase of only 0.7 ft. on the Cairo gauge. It is apparent, therefore, that if the crevasse had not occurred at Dorena, and making the usual allowance for the additional rise that would ordinarily have resulted from the high stage at St. Louis, the crest stage at Cairo would have been approximately 58 ft. about, but not after, the end of April. Incidentally, with the stages at Cairo and St. Louis as they were on the morning of April 16, and no crevasse at Dorena, but with the Upper Ohio in flood equal to that of March, 1913, the crest stage at Cairo would probably have approximated 62 ft.

Finally, it is within the bounds of possibility that the indicated stages in Table 11 could be exceeded. It can be shown, for instance, that stages of 65 ft. at Cairo, and of 73 ft. at Arkansas City, are not entirely removed beyond the limits of possibility under the most favorable conditions of precipitation distribution and synchronization. However, the probability is so very remote that its complete realization can perhaps be disregarded, as the sequences of the proper meteorological combinations that would be necessary to produce the extreme conditions as computed, while not absolutely impossible, are almost beyond belief.

RUN-OFF CHARACTERISTICS OF THE MISSISSIPPI RIVER DRAINAGE BASIN

By Nathan C. Grover,* M. Am. Soc. C. E.

Flood problems in general can be presented most satisfactorily to engineers and the interested public in the simple terms of quantities of water and capacities of channel. In such simple terms the flood problems of the Mississippi may be said to consist in determining the quantities of water to be expected in various stretches of the river and in making ample provision in channel and temporary storage capacities for conveying those quantities to the Gulf. If these problems are to be thus set forth, it is necessary that reliable continuous records of discharge be collected for the critical stretches of the Mississippi River below Cairo, Ill., and for its major tributaries. Unfortunately, such records, covering the crests of floods and showing the possibilities of their synchronization, have not been made, and this basic information is, therefore, not now available.

A few approximate figures may serve to make the situation clearer. A typical flood at Cairo, consisting of the combined discharges of the Ohio River and the Mississippi above the Ohio, is 2 000 000 cu. ft. per sec., more or less. The tributaries below Cairo may add to such a flood enormous quantities of water, amounting in 1927 to more than 1 500 000 cu. ft. per sec., according to the best estimates that have been available to the writer, making an apparent flood discharge of more than 3 500 000 cu. ft. per sec. for which channel and storage capacities must be provided so far as they do not already exist. The maximum capacity of channel at New Orleans, La., with the levees intact is about 1 500 000 cu. ft. per sec. Any satisfactory plan of flood control must afford the means for handling the water in the lower river in excess of the channel capacity at New Orleans by providing either additional channels to the Gulf, definitely set aside for that purpose, or sufficient lateral storage capacity to detain the excess water until it can be discharged through present recognized channels, or, more probably, channel capacities to carry the major part of the floods and supplementary lateral storage to take care of the remainder.

The writer does not wish to be understood as stating that the discharge of the Mississippi in 1927 reached in any stretch at any time the enormous total of 3 500 000 cu. ft. per sec.; some channel and lateral storage had been left available to the river, many crevasses were made whereby the river re-occupied its natural lateral basins that had been closed by levees, and there was a great quantity of water that broke through the artificial barriers that had been constructed and re-occupied the natural channels across the delta to the Gulf, all of which, by returning in part to Nature's plan, tended to decrease the crest discharges in the lower river, which have been aggravated

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by the very works designed to control them. In spite of these mitigating conditions, however, the waters in this flood covered nearly 20 000 sq. miles, situated in seven States, an area approximately equal to that of New Hampshire and Vermont combined.

A comparison of the Mississippi with other great rivers of the Continent will serve to emphasize the magnitude of the quantities of water to be controlled in a Mississippi flood. Available estimates indicate that the total run-off in a great flood of the Mississippi, covering a period of six months or more, may exceed 500 000 000 acre-ft.—a quantity equal to the average annual run-off of that river or to the total run-off of either the Columbia or the St. Lawrence for three years, or of the Colorado for more than thirty years. The seven largest reservoirs constructed by the U. S. Bureau of Reclamation in the Western States have an aggregate capacity of 7 280 000 acre-ft., or about 1½% of the estimated 500 000 000 acre-ft. in a single Mississippi River flood. The proposed Boulder Canyon Dam, with a capacity of about 25 000 000 acre-ft., would store the total run-off of the Colorado for nearly 1½ years, and yet this capacity, great as it is, is equivalent to only 5% of the run-off in a single flood of the Mississippi.

The area from which the water of the Mississippi is derived, aggregating 1 240 000 sq. miles, is widely diversified in topography, geology, vegetation, and climate. It constitutes the great central basin of the country, extending from the crest of the Alleghanies to the crest of the Rockies and from the Great Lakes to the Gulf of Mexico, and includes mountains and plains, forests and grasslands, fields that are cropped annually, and remnants of the great public range. This great area includes all or parts of 31 States and of 2 Provinces of Canada, and is equivalent to nearly 41% of the area of the Continental United States. It is pre-eminently a great food-producing region and contains about 42% of the population and 39% of the wealth of the country. No other river basin on the globe combines topography, soil, and climate to make attractive living conditions for so many people as that of the Mississippi. It has within its borders many cities, railroads, highways, and farms with artificial or modified drainage conditions. These works of man, which have certainly affected the rate and probably the total quantity of discharge of the Mississippi, cannot be decreased, however desirable it might be in the interest of flood control to restore the basin to the conditions of Nature. In fact, to restore it to such conditions would eliminate the need for flood control, because no one would be there to be damaged. Rather, it is certain that such works will be continually increased with the growth in population, industry, transportation, and wealth of the country. It may be possible, however, to improve present conditions by a reasonable program of forestation and by the construction of reservoirs in head-water streams, but, in the writer's opinion, it is vain to hope for a material reduction of the flood flows of the Lower Mississippi by such methods. Although it is recognized that vegetation, including forests, affects run-off and that reservoirs on head-water tributaries may be operated so as to decrease in a small way the floods in the lower

river, the quantities of water in a Mississippi flood are so great that these two influences, which will always be considered beneficent, can probably never have a major effect and at most can only add some small amount to a reasonable factor of safety which should be contained in any plan of flood control.

The Mississippi above the mouth of the Missouri and the four principal tributaries—the Missouri, Ohio, Arkansas, and Red—drain 96% of the total area of the Mississippi Basin. There are gauging stations which will yield continuous and reliable records at all stages of discharge from about 86% of the area drained by rivers above Cairo. Below Cairo, however, the run-off from only 18% is fully measured, and this 18% does not include any appreciable part of the valley that is subject to torrential rains.

There are several long-time records of the stage of the Mississippi between Cairo and the Gulf, and many measurements of discharge have been made by current meters and floats whereby some of the gauges have been fairly well rated for conditions of unbreached levees. There are, however, in this part of the basin no records of stage that can be satisfactorily converted to records of discharge for the varying conditions of slope and of flow through lateral flood channels resulting from crevasses. In other words, there are no reliable records of crest discharge of either the main river, or of its tributaries below Cairo, and the existing voluminous records are scattered through hundreds of reports, in part out of print, and are too lacking in adequate analysis, interpretation, and co-ordination to be reasonably usable.

The eastern and northern tributaries drain regions of moderate precipitation that accumulates in part during the winter in the form of snow and ice, which thus form the source of the characteristic spring freshets that occur when the rising temperatures release the water. The head-water feeders of the southern, western, and northwestern tributaries drain large regions of semi-aridity, in which the precipitation decreases gradually toward the west. More than one-third of the basin of the Mississippi lies west of the 100th Meridian which passes through the middle of the Dakotas, Nebraska, and Kansas. This part yields considerably less than 5% of the water of any flood. By contrast, the Ohio, draining 17% of the basin, may yield 50% or more of a Mississippi flood, and the lower basins of the southern tributaries and the lower valley of the Mississippi itself, which are subject to heavy rains that may occur in any month with the intensity of cloudbursts, yielding several inches of rain in 24 hours, may furnish a considerable and critical part of the flood waters.

An illustration of the differences in run-off between the humid and dry parts of the western and southern tributaries is afforded by the recorded run-off of the Missouri River for the year October 1, 1925, to September 30, 1926, at Leavenworth, Kans., and at Boonville, Mo. At Leavenworth, with a drainage area of 428 000 sq. miles, the run-off was 26 400 000 acre-ft.; at Boonville, with a drainage area of 508 000 sq. miles, or only 20% more than that at Leavenworth, the run-off was 40 900 000 acre-ft., or nearly 55% more.

The foundation or base of every great flood in the Mississippi comes from the Ohio. Starting with at least a reasonably large flood in the Ohio, the

building up of a great Mississippi flood depends on the synchronizing of that flood with floods from other tributaries as the crest progresses toward the Gulf. The peaks of the Ohio floods generally pass into the Mississippi between January 1 and May 31. They frequently join with high water from the Upper Mississippi at Cairo, as the records of stages at St. Louis show that April, May, June, and July are the months in which crest stages generally occur at that place. Fortunately, however, for floods at Cairo, the crest stages from the Ohio are generally ahead of the crest stages from the Upper Mississippi, but as crest stages may occur in either river in April and May, there is a possibility that in some years the crests of an early flood from the Upper Mississippi and a late flood from the Ohio may reach Cairo simultaneously. Nevertheless, it is probable that the climatic conditions that cause early floods on the Upper Mississippi also cause them on the Ohio—in other words, that climatic conditions make the possibility of a combination of crests of floods from the two rivers at Cairo rather remote.

If there were no influx of flood waters below Cairo, the flood problems of the Lower Mississippi would probably be relatively simple; because the natural channel and flood-plain storage would so flatten the crests of the floods passing Cairo as to permit the water to pass without causing damage through those stretches of channel in the lower river that have smaller capacities than the channel just below the mouth of the Ohio.

The yield of the lower tributary basins and especially of the parts of those basins that lie within 200 miles on either side of the Mississippi, in the path of the big storms, is, therefore, of great importance in any study of floods, and it is just this part of the run-off that is largely unknown. The contribution of the lower part of the basin to the disastrous floods of the Mississippi is caused exclusively by heavy rains that occur most frequently from October to March, but that come occasionally in the late spring and early summer. The rate of local run-off is dependent on the magnitude and intensity of the precipitation over this southern part of the basin. The probability of floods in the Ohio synchronizing with floods in the lower tributaries of the Mississippi appears, therefore, to be much greater than that of floods in the Ohio synchronizing with floods in the Upper Mississippi.

The floods of the Mississippi are thus caused in the first instance by floods in the Ohio to which melting snows in the head-water tributaries contribute, but with which must be combined high water from the Mississippi above Cairo or floods in the southern tributaries due to heavy rains over broad areas of the lower basin, or by some combination of these conditions. The combination of climatic conditions that produces floods has occurred periodically for countless ages and will be repeated again and again as years go by, and floods will surely recur with those conditions. De Soto experienced a great flood when he visited the Mississippi, but it caused no great damage because there was little to be damaged. Future floods may be no greater than that of De Soto's time, but if they are uncontrolled they will be increasingly destructive in lives and property as the use of the great flood-plains of the delta increases.

The flood-plains of the Mississippi and its delta have been built by its floods. The lands that are now protected by levees from overflow owe their very existence to floods. The process of valley and delta building involves periodic inundation and the deposition of silt. River channels through the flood-plains and delta do not normally have sufficient capacities to carry within their banks the floods that occur. A change from this fundamental condition of overtopped banks and temporary lateral storage of water is, therefore, man-made and represents a radical departure from the natural processes of a silt-bearing river that is constantly building its delta. It must be recognized that such an artificial change in the regimen of a great river is always attended by difficulties and dangers and that to be adequate, it must be on a scale commensurate with that of the natural forces.

In Nature's process of flood-plain building, a Mississippi flood, after leaving Cairo, would normally spread out over several thousand square miles of bottom-land included in the St. Francis, Tensas, Yazoo, and other basins, and would be drained therefrom over a period of several months at a fairly uniform but lesser rate than the maximum inflow to those basins. Consequently, the channels leading from the basins to the Gulf would be formed with smaller capacities than the channels discharging water into the basins. So long as the great overflow basins were available for temporary storage of excess flood waters, the smaller channel capacities across the lower delta were ample. As man has gradually shut off these natural regulating basins by building levees, thereby depriving the river of their effects in equalizing flow, the floods below the basins have inevitably increased in magnitude, and greater channel capacities have been required. Available records of stage show clearly this situation, which should have been expected as surely as increased fluctuations in the stage and discharge of the St. Lawrence would follow if river channels were to be constructed through the Great Lakes so as to eliminate the equalizing effects afforded by the natural storage of those wonderful natural basins.

The run-off characteristics of the Mississippi Basin above Cairo are fairly well known and are not being changed radically by man, but those below Cairo have been and are being gradually changed with progress in the construction of levees, and there are no satisfactory records of the resulting changes in crest rates of discharge. The run-off characteristics will continue, of course, to change until stability has been reached in a system of levees, spillways, and overflow basins that will carry uniformly the successive floods of the river.

Students of hydrology have long realized that a flood like that of 1927 was inevitable. They know, too, that similar and even larger floods will occur in the future. To the end that the magnitude and periodicity of these certain great floods may be known within reasonable limits of error, it is necessary that the yield of water by the tributaries shall be definitely known over a period of years, not only in total quantities, but in distribution throughout the year and in variations from year to year. On no other basis can satisfactory forecasts of future floods be made as a necessary first step in designing control works.

The re-occupation of leveed basins and the breaking of barriers to natural overflow channels to the Gulf cannot, of course, be longer left to chance, but such capacities of channels and overflow basins must be provided in advance of flood as to remove the danger of disastrous breaks in levees. It may be stated, as a self-evident truth, that the nearer the methods of flood control conform to Nature's way the less will be the difficulty in maintaining the control works, and, conversely, the greater the departure from Nature's way the stronger will be the tendency for the river to return to natural conditions and the more difficult will be the maintenance of these works.

It is the writer's opinion that an adequate determination of the necessary channel and storage capacities can be made only on the basis of reliable estimates of the quantities of water for which provision must be made in the several stretches of the river, and such estimates must rest on continuous reliable records of discharge. Similarly, the problem of selecting the best locations for temporary storage of the flood waters in lateral basins and the most practicable auxiliary channels to the Gulf cannot be intelligently studied or correctly solved until the area subject to floods has been topographically mapped. As less than 8% of this area is now mapped, the problem is, indeed, difficult.

It is fortunate that the same records and maps will serve as a basis for both flood control and general development. The problems arising in connection with the utilization and administration of surface waters in the Mississippi Valley are constantly increasing. The best solution of many of these problems can be reached only on the basis of adequate data in regard to quantities of run-off and its distribution with respect to both time and drainage areas. As utilization progresses, these data are becoming more essential and are increasingly in demand by a wide range of users, including Federal, State, and municipal officials, corporations, and individuals. Definite plans for flood-control works and for important structures to utilize rivers, or to be built on their banks or flood-plains, should not, in the interest of safety and economy, be adopted in the absence of adequate hydraulic investigations and hydrologic data, or of complete topographic maps.

The many failures of enterprises developed on the basis of insufficient records of water supply show conclusively that it is unsafe to proceed with developments involving the use of rivers except on the assurance afforded by adequate records of discharge. That estimates of water supply made from records of rainfall and comparisons of drainage areas may be unreliable and misleading has been demonstrated repeatedly, and the Mississippi Valley is no exception to the general rule. The wise development of this valley must rest on adequate data in regard to the discharge of its many rivers. Because of the diversity of interests, of which flood control is doubtless the most pressing, and the complexity and interrelation of the many problems involved, provision should be made for the collection of these necessary data on a continuing basis, and for their analysis, correlation, and publication promptly, in order that they may be available for use by engineers and the public.

The floods of the Mississippi affect the development and prosperity of a region that is of major importance, not only to the United States, but to the world. As population increases and development progresses, the resources of this vast region must ultimately be completely utilized in the production of foodstuffs and textile raw materials needed by the world. Whatever effects the levees and other works for controlling floods may have on the run-off characteristics of any part of the basin, the control must be accomplished in the interest, not only of the flooded regions, or the entire Mississippi Basin, but of the whole country. Full justice cannot be done if the situation is approached with a narrow-angle view and near-sighted vision.

THE WORK OF THE MISSISSIPPI RIVER COMMISSION

By C. W. Kutz,* M. Am. Soc. C. E.

The Mississippi River Commission was created by an Act of Congress approved June 28, 1879. It consists of seven Commissioners, three of whom are selected from the Corps of Engineers, U. S. Army; one from the U. S. Coast and Geodetic Survey; and three from civil life, two of whom must be civil engineers.

The basic law required the Commission to "mature a plan or plans as will correct, permanently locate and deepen the channel, and protect the banks of the Mississippi River; improve and give safety and care to the navigation thereof; prevent destructive floods; promote and facilitate commerce, trade and the postal service".

It also called for a report in full on the practicability, feasibility, and probable cost of the various plans known as the jetty system, the levee system, and the outlet system, as well as on such others as the Commission deemed necessary. Before presenting the conclusions of the Commission as embodied in its first report, it seems desirable to describe briefly the physical characteristics of the Lower Mississippi Valley and the work that had been done prior to the creation of the Commission and, particularly, to refer to the prior Federal investigations and surveys, as on the results of those studies the plan of the Commission was largely based.

Physical Characteristics

The Mississippi River and its tributaries drain an area of 1 240 000 sq. miles including all or portions of 31 States and about 20 000 sq. miles of Canada.

The natural divisions of the basin are as shown in Table 12.

TABLE 12.—Divisions of the Mississippi Basin.

Designation.	Area, in square miles.	Ratio to entire basin.
Upper Mississippi Basin	165 900	0.13
Missouri Basin	527 100	0.43
Ohio Basin	201 700	0.16
Arkansas Basin	186 300	0.15
Red Basin	90 000	0.07
Central Valley	69 000	0.06
Total	1 240 000	1 00

The alluvial valley, with which the Commission is chiefly concerned, begins at Cape Girardeau, Mo., 52 miles above Cairo, Ill. This flood-plane of alluvial deposit varies in width from 20 miles just north of Natchez to 80

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miles at Greenville, Miss., the average width above Plaquemine, La., being 45 miles. Below Plaquemine it spreads out into a broad delta, the general shape of which is a semi-circle having a radius of about 80 miles.

Through this valley the river winds its way to the Gulf, generally following the bluffs on the east from the mouth of the Ohio to Memphis, Tenn., touching the west bluffs at Helena, Ark., and hugging the east bluffs closely from Vicksburg, Miss., to Baton Rouge, La. The air-line distance from Cape Girardeau to Head of the Passes is about 600 miles, the river distance, 1 102 miles.

The river flows in a well-defined but unstable channel, the natural banks of which are from 3 to 12 ft. above the general level of the plain. These banks are highest nearest the river and have an average slope away from the river of 7 ft. per mile in the first mile. The river is constantly eroding its banks in unprotected bends and forming new land on points. This caving is accentuated in many places by an underlying strata of sand which washes out and causes the bank to fall by its own weight as the river subsides.

Above the mouth of Red River on the right bank, and above Baton Rouge on the left bank, the drainage of the valley finds its way into the river through tributaries. Below these points the drainage is carried away from the river and into the Gulf of Mexico through numerous bayous and streams.

The minimum measured discharge of the river at Columbus, Ky., below the mouth of the Ohio, is 71 000 sec-ft.—the maximum at this point, 2 015 000 sec-ft. having occurred in 1912. The carrying capacity of the natural channel at bank-full stage is about 1 000 000 sec-ft.

The building of levees on the banks of the Mississippi River began with the first settlements in the lower alluvial valley (as early as 1717), and as a measure of economy they were built on the highest ground, that is, near the river bank. The extension of levees kept pace with the establishment and growth of settlements along the lower river, each planter being required to complete the levee along his own river-front. In 1812, when Louisiana was admitted to the Union, the levees extended on both sides of the river, to Baton Rouge on the left bank and to Point Coupee on the right bank. By 1844, they were almost continuous from 20 miles below New Orleans to the mouth of the Arkansas River on the right bank and to Baton Rouge on the left bank.

The floods of 1849 and 1850 caused widespread damage in the valley. National interest was aroused and in response to a plea for Federal aid, the Swamp Land Act was passed in 1850. By this Act the Federal Government granted to the several States in the Lower Mississippi Valley all unsold swamp lands and overflowed lands within their limits to provide funds to reclaim the districts subject to overflow. The States of Louisiana, Mississippi, Arkansas, and Missouri organized offices for the sale of swamp lands and appointed Commissioners for the location and construction of levees. Due, however, to differences in laws governing levee construction, to differences between counties and parishes within a State, and to a total lack of co-ordination and co-operation, but little effective protection was secured from the proceeds of the sale.

In the same year, Congress appropriated $50 000 "for a topographical and hydrographical survey of the delta of the Mississippi with such investigations as may lead to determine the most practical plan for securing it from inundation * * *". This work was placed in charge of Capt. A. A. Humphreys, Corps of Engineers, U. S. A. Field work was started in 1851, but suspended in the fall of that year until 1857 when it was resumed by Lieut. Henry L. Abbot, Corps of Engineers, U. S. A., who had been detailed as Assistant to Captain Humphreys. The final report on this survey was submitted jointly by these officers in 1861.*

This report which is very comprehensive, discusses as plans for protection, a system of cut-offs, the diversion of tributaries, a system of head-water reservoirs, and a system of outlets as alternatives for levees. Its conclusions were that no advantage could be derived from diverting tributaries or constructing reservoirs, that the plan of cut-offs and new or enlarged outlets to the Gulf were too costly and too dangerous to be attempted, and that levees could be relied upon for protecting all the alluvial bottom-lands liable to inundation below Cape Girardeau. It noted, however, the possibility of reducing levee heights near Lake Providence, La., by constructing near that town an outlet leading to Bayou Tensas and the Black River, but its approval of this alternative was conditional upon further investigation.

The Humphreys and Abbot investigation included measurements for the determination of the quantity of sediment in suspension in Mississippi River water. These observations were begun at Carrollton, La., in 1851, and continued for two years and the results, combined with similar measurements made by other agencies, led to the following conclusions:

(*a*) "* * * the Mississippi water is not charged to its maximum capacity with sediment; * * * A most important practical deduction may be drawn from this fact, namely, the error of the popular idea that a slight artificial retardation of the current, that caused by a crevasse, for instance, must produce a deposit in the channel of the river below it.

(*b*) "* * * the sediment of the Mississippi is to the water, by weight, nearly as 1 to 1 500, and by bulk, nearly as 1 to 2 900; provided long periods of time be considered. * * * and if the mean annual discharge of the Mississippi proper be correctly assumed at 19 500 000 000 000 cubic feet, it follows that * * * sedimentary matter, constituting one square mile of deposit 241 feet in depth are yearly transported in a state of suspension into the gulf.

(*c*) "Besides the amount held in suspension, the Mississippi pushes along into the gulf large quantities of earthy matter.

"No exact measurement of the amount of the annual contributions to the gulf from this source can be made, but from the yearly rate of progress of the bars into the gulf * * * it appears to be * * * which would cover a square mile about 27 feet deep.

"The total yearly contributions from the river to the gulf amount then, to a prism 268 feet in height, with a base of one square mile. * * *"

The next milestone in the history of the Lower Mississippi was an Act of Congress (1874) appointing a Joint Civil and Military Commission to make a full report upon the best system of permanent reclamation and redemption of the alluvial valley from inundation. The report of this Board, submitted in

* "Report upon the Physics and Hydraulics of the Mississippi River," 1861.

1875, was based upon the results of the Humphreys and Abbot survey, and upon additional data secured subsequent thereto. Its conclusions were: "No practicable aid can be derived from any diversion of tributaries or by artificial reservoirs; that cut-offs are very pernicious and that outlets, though correct in theory, find no useful application to the Mississippi". A general system of levees was recommended.

The Mississippi River Commission was created four years later. In its first report to the Secretary of War (1880), it advocated as the best means of securing the needed improvement in its navigation, a plan which would concentrate, rather than disperse, the waters of the river. In this report it also condemned the outlet system as a means of flood control, expressing the belief that "no surer method of ultimately raising the flood surface of the river can be adopted than by making lateral outlets for escape of its flood waters."

Asserting that the bad navigation of the river was produced by the caving and erosion of its banks, the excessive widths, and the bars and shoals resulting directly therefrom, it proposed to narrow the river, where too wide, by contraction works and to protect the caving banks by bank revetment. This was assumed by the Commission to be the plan referred to in the Act as the "Jetty System". As a means of protection against floods, it recommended levees as essential to prevent destruction to life and property by overflow. The Commission was not agreed as to the value of levees as an auxiliary to a plan of channel improvement.

The first appropriation for the improvement of the Mississippi River in accordance with the plans of the Commission was made in March, 1881, and work continued for several years, but several facts were soon developed, namely, that the light contraction works originally designed were much too weak; that the cost of the improvement would be much greater than had been originally estimated; that one of the first results of contraction work was to increase bank caving; and that generally bank revetment was of greater relative importance than had at first been considered.

The work was interrupted in 1885 by a legislative proviso that no works of bank protection or revetment should be executed until it was found that permeable contraction works would not secure the desired stability of the river banks. This prohibitive clause was not revoked until late in 1888 so that four successive working seasons were practically lost. After that date, bank protection became of prime importance, contraction works being restricted to the repair and completion of work under way.

Although the Commission in its report of 1880 had recommended levees as a means of protection against floods, Congress was then unwilling to expend Government funds in protecting the land of private owners, for the first Appropriation Act contained the following proviso:

"That no portion of the sum hereby appropriated shall be used in the repair or construction of levees for the purpose of preventing injury to land by overflow or for any other purpose whatever except as a means of deepening or improving the channel of said river."

In taking this position Congress was apparently influenced by the fact that in 1850 it had, through the passage of the Swamp Land Act, conveyed to the States a free title to all the unsold swamp and overflow lands in the delta basin with the understanding that they would be sold and the proceeds used to protect them against overflow. In the Appropriation Act of 1882 it was again provided:

"That no part of this appropriation shall be expended for repair or to build levees for the purpose of reclaiming land or preventing injury to land by overflows; provided that the Commission is authorized to repair and build levees if in its judgment it should be done as a part of its plan to afford ease and safety to the navigation and commerce on the river and to deepen the channel."

Under the strict prohibitive clause in the Act of 1881 no part of the appropriation was allotted to levees. Under the Act of 1882 which gave somewhat wider discretion to the Commission, a large part of the appropriation was allotted to levee construction. From that time on the policy of allotting money for levee purposes was followed, but it was not until 1890 that the prohibitive clause was removed.

From 1890 to 1917, the periodic appropriations made by law for expenditure by the Commission were divided between levees, revetments, and dredging, but the work on the levees was looked upon as an adjunct to river improvement. It was not until the Flood Control Act of 1917, that flood control became by law as definitely a part of the Commission's work as is river improvement. That Act provided for co-operation by local interests, they to provide the necessary right of way and to pay not less than one-third of the cost of levee construction, and to assume the entire cost of maintenance after completion. The cost of river improvement (revetment and dredging) was to be paid for entirely out of Federal funds.

Considered as an adjunct to river improvement, it was natural that levees should be placed fairly close to the bank. This placed them on the highest ground, thereby reducing their cost, it prevented cross-currents and consequent fill within the river banks during flood, to the detriment of the low-water channel, and helped to stabilize the main channel of the river. Even from a flood-control standpoint there is much to be said in favor of placing the levee line fairly close to the river bank.

Levees

The levee section of 1882 was a modest earth embankment with an 8-ft. crown and slopes of 1 on 3, with practically no free-board above the flood of that year. At Lake Providence this resulted in a section that contained 161 sq. ft., or 31 000 cu. yd. per mile. As the levee lines were extended and the flood flows more and more confined, it became necessary to raise the levee grade and correspondingly increase the section so that, in 1896, there was at Lake Providence a cross-sectional area of 530 sq. ft. and a yardage of 103 000 per mile, and, in 1914, a cross-sectional area of 2 158 sq. ft. and a yardage of 422 000 per mile.

This gradual building upward, toward the levee grade required to confine a given flood completely, is believed to be responsible for the widespread fallacy that when a sedimentary stream like the Mississippi is confined between levees, levee grades must be steadily raised to keep pace with a rise in the river-bed. In some instances it has been asserted that the bed of the Mississippi River, due to the levees, has risen until it is above the surrounding country. This view, that there has been a marked change in the elevation of the river-bed, is not borne out by the results of surveys made at many different points over a long term of years.

In 1924, a study was made of a reach of the river along the Lower St. Francis Basin in Missouri and Arkansas. This basin has a river frontage of 236 miles, from New Madrid, Mo., to Helena, Ark. The construction of modern levee systems in that section was not begun until 1893, except for about 60 miles on the east bank of the river. The period considered was from 1882 to 1921, inclusive, a period extending from no levees to complete levees. The low-water troughs of each year passing the gauges in the reach were tabulated to show the elevations of such troughs at the various gauges, all of which had been established prior to 1882 and had been read regularly since that date. This comparison showed a decided lowering of the low-water plane.

Recently, more than 2 000 measured cross-sections at different points on the river were compared to show the changes that had occurred between early surveys and those of recent years, and the results show uniformly a moderate but well-defined increase in width, depth, and cross-sectional area. It is a well-recognized fact that Mississippi River bars do build up on a rising river and scour out on a falling river, but these are seasonal changes and the effects are not permanent. Bars also travel down stream, diminishing the depth on one section and increasing it on another; but there is no evidence whatever in the records of the Mississippi River Commission to support the view that the bed of the river is being raised due to the construction of levees.

Prior to the flood of 1927 the major flood of record was that of 1912, and, as a result of that flood, the Commission established levee grade lines along the main river computed to be high enough to contain with a 3-ft. free-board a flood of equal magnitude. At the same time it established new standard levee sections defined as follows:

"Levee: Crown to be 8 ft. wide—reduced to 6 ft. above mouth of Missouri River; front slope to be 1 on 3 to intersection with natural ground (or bottom of old borrow-pits where enlargements have thrown the toe of the levee so far forward); and back slope to be 1 on 3 to intersection with top of banquette, which will be from 5 to 8 ft. below the levee crown.

"Banquette: Top to be 20 ft. wide for levees from 10 to 13 ft. high, 30 ft. wide for levees from 13 to 16 ft. high, and 40 ft. wide for levees exceeding 16 ft. in height; top to have slope of 1 on 10 away from the levee; and rear slope to be 1 on 4 to intersection with natural ground. In cases of extreme height —crossing sloughs or bayous—a false berm may be required beyond the rear slope of the banquette."

Subsequently, slight modifications were authorized by the Commission in both grade and section but, generally speaking, both were substantially unchanged at the time of the flood of 1927. Although work had been in progress

for thirteen years, two more years would have been required for completion of the levee project at the rate at which funds were being appropriated.

The Annual Report of the Commission for 1927 shows a total expenditure to date for levees by the United States from Federal funds of $71 000 000 and an expenditure by the United States from funds contributed by local levee districts of $15 000 000. In addition to this expenditure of $86 000 000 by the United States, State and local organizations, prior to and since the creation of the Mississippi River Commission, expended approximately $152 000 000 on levees and in expenses incident thereto, such as rights of way, repair work, high-water expenses, etc. This combined expenditure of $238 000 000 is represented by 1 880 miles of levees containing approximately 500 000 000 cu. yd. of material. This shows an expenditure of slightly less than 50 cents per cu. yd. of material now in the levees; the difference between this and actual cost per cubic yard being represented by lost or abandoned levees, by expenses for maintenance and repair, and in the cost of closing crevasses and fighting floods.

When Commission reports are consulted, progress toward completion of the 1914 levee project seems slow and the amount of levee lost or abandoned seems large. One reason for the rate of progress lies in the fact that the Commission's jurisdiction on tributaries has been gradually extended, so that, while in 1918 the estimated final contents of the levee line was 473 000 000 cu. yd., in 1927 it was 534 000 000 cu. yd., although no change in grade or section had been made.

In six years, from 1918 to 1924, the lost or abandoned yardage amounted to 15 800 000, about 0.7 of 1% per annum of the average annual contents of the system. The losses have been due partly to the necessity of abandoning old lines where threatened by caving banks and partly to changes in alignment in order to secure better foundations. The caving of banks is much the greater cause of losses, and bank revetment is the only remedy for this evil.

Bank Revetment

A number of years ago it was estimated that, of the 2 000 miles of bank-line between Cairo and the Head of the Passes, more than 700 miles were in need of revetment. The funds available for such work, however, have never been sufficient to justify undertaking the work on a large scale or in a systematic manner. Each year it has been necessary to confine revetment work to cases of the most urgent necessity, such as threatened cut-offs or endangered levees that could be replaced only at excessive cost. Even if the work has of necessity been prosecuted in this haphazard manner there was in place, at the end of the fiscal year 1927, 128 miles of revetted bank.

The revetment in most common use is made of small willow trees from 2 to 6 in. in diameter and from 30 to 50 ft. long, which grow rapidly and in abundance on the sand-bars along the river. In the earlier forms, the revetment was composed of these small trees, laid diagonally, in two layers, and the mass given a certain rigidity and strength by means of large "poles", below and above, tied together by galvanized wire. This type has given way to the

"fascine" mattress, composed of small bundles of young willows bound by galvanized wire. These fascines are placed side by side to make up the length of the mattress, and are held in position by being tied underneath by means of galvanized "sewing strands" running over and under the bundles to longitudinal cables. In the earlier forms of mattress, these fascines were made complete, and were bound in the circular form by short lengths of wire, before being placed in the mattress. In the present mattress, the two processes are combined. Longitudinal wire cables and larger trees or "poles" give the requisite strength and rigidity.

These mattresses are built on floating ways in lengths of 1 000 ft. and of varying widths depending upon the character of the bank to be revetted. The mattresses are sunk in position by stone dumped from barges. In certain places where rock was costly, concrete ballast blocks weighing about 65 lb. were used instead of rock.

A number of different types of reinforced concrete mattresses have also been used, one of them, developed in the Third Mississippi River Commission District, having been in use about five years. This type of revetment is fabricated in units 4 ft. wide by 25 ft. long and 3¾ in. thick, each unit consisting of 25 concrete slabs 1 by 4 ft. by 3¾ in., reinforced and held together by wire reinforcement. These units are cast on deck-barges and, as soon as the concrete has hardened sufficiently, a second layer is cast on top of the first, the layers being separated by heavy paper. These units are united by cables into mattresses on floating launching ways and are sunk with their long side perpendicular to the bank. The willow mattresses are constructed with the long edge parallel to the bank. Since its inception this type of revetment has been improved by increasing the size and, therefore, the life of the reinforcement and by increasing the thickness of the slabs from 3 to 3¾ in., but sufficient time has not elapsed to determine fully the relative durability of concrete and willow mattresses. The concrete mattresses, being smaller in size and with the long axis perpendicular to the bank, more readily adjust themselves to inequalities in the river bank than the longitudinal willow mattresses and, furthermore, they are nearly 20% cheaper.

For certain reaches of the river this type of concrete revetment is believed to be too light. For use in such places experiments are in progress with a mattress composed of overlapping and interconnecting concrete slabs 4 in. thick, 6 ft. wide, and 11 ft. long. A large scale experiment with this type of revetment is now in progress.

Mattresses, whether of willow or concrete, protect only the under-water slope, extending out from the water line to about 50 ft. beyond the foot of the slope. Above the water line the bank is graded to a slope of 1 on 3 and paved with rip-rap stone, or concrete.

To the end of the last fiscal year the Commission had expanded in new revetments, and in repairs and renewals, $61 000 000, an average of $475 000 per mile of effective revetment. In addition to its expenditures in levees and revetments, the Commission is maintaining and operating a fleet of dredges

at an average annual expenditure of $600 000, in order to maintain a navigable depth of 9 ft. over bars during low-water periods.

As revetment work progresses the material annually caved into the river will decrease, and it is anticipated that this decrease in the amount of material in motion will greatly improve the low-water channel and decrease expenditures for dredging.

The levee work is primarily in the interest of flood protection and, as was mentioned previously, is paid for in part by local interests. The dredging work is in the interest of navigation and, like most other navigation projects, is paid for wholly by the Federal Government.

The revetment work on the main river is essential to both flood protection and to navigation, but under existing law it is regarded as part of the navigation project and paid for wholly by the United States, except in one or two places where local levee districts have voluntarily paid for such work in order to supplement what the Commission was able to do.

In appraising the work of the Mississippi River Commission it should be borne in mind that it has been working toward a limited objective at a rate fixed by Congress. It was never assumed by the Commission, nor by Congress, that its plan would do more than protect against a flood equal to that of 1912. In working toward this limited objective it has many times been importuned to provide special relief for New Orleans by the construction of one or more spillways above or below New Orleans so that that city might be spared the expense of raising its wharves to a height compatible with the 1914 grade line established for the entire river. Such work the Commission has heretofore declined to undertake on the ground that the limited funds at its disposal should be used in giving a uniform measure of relief throughout the valley and that to divert $5 000 000 or $10 000 000 to the construction of spillways at or near New Orleans would be unfair to those levee districts the levees of which were still not up to the grade line of 1914.

Flood of 1927

The rainfall throughout the valley of the Mississippi River which caused the flood of 1927 was extraordinary in volume and in duration and it occurred at a time when the natural storage in the Mississippi itself and the storage basins contiguous to it in the delta were already well filled. The rainfall in Eastern Oklahoma, Western Arkansas, and Southern Missouri was unprecedently heavy and caused discharge from the Arkansas River almost as great as that of ordinary high water in the Mississippi itself.

It is difficult to compare the magnitude of different Mississippi river floods at a given point on the basis of maximum measured discharges alone, for at the same gauge height the discharge will vary widely due to differences in slope, and the slope, in turn, will depend upon the extent to which the river basin is filled and upon the rate of rise.

Columbus, Ky., 22 miles below the mouth of the Ohio River, at Cairo, is a gauging station, and discharge measurements made there or in that vicinity are available for each flood-year since 1892. As a check on the observed dis-

charges it is customary to assemble the volume contributed by the Mississippi and the volumes contributed by the Ohio at Evansville, Ind., and its principal tributaries below that point. Table 13 shows the volume of discharge (as scaled from rating curves) contributed by tributaries of the Ohio and by the Ohio River as a whole. The greatest discharge that occurred within a period of 3 to 6 days before the flood crested at Cairo was used for all stations. The discharge at the maximum recorded stage for each station is also given.

TABLE 13.—VOLUME OF WATER CONTRIBUTED BY OHIO RIVER AND ITS TRIBUTARIES, IN CUBIC FEET PER SECOND.

River	Wabash.	Ohio.	Cumberland.	Tennessee.	Ohio.*
Station	Mt. Carmel, Ind.	Evansville, Ind.	Nashville, Tenn.	Florence, Tenn.	Total.
Year.					
1890	140 000	572 000	176 000	341 000	1 229 000
1892	90 000	471 000	114 000	207 000	882 000
1893	190 000	475 000	34 000	185 000	884 000
1897	143 000	356 000	166 000	575 000	1 240 000
1898	286 000	615 000	52 000	126 000	1 079 000
1903	134 000	551 000	113 000	218 000	1 016 000
1904	283 000	501 000	79 000	131 000	994 000
1907	171 000	623 000	72 000	52 000	918 000
1912	140 000	552 000	148 000	270 000	1 110 000
1913	346 000	665 000	146 000	238 000	1 395 000
1916	164 000	526 000	72 000	107 000	869 000
1917	52 000	550 000	102 000	290 000	994 000
1920	76 000	551 000	64 000	178 000	869 000
1922	190 000	560 000	109 000	278 000	1 137 000
1927	100 000	435 000	79 000	200 000	814 000
Maximum recorded..	400 000	673 000	235 000	596 000	....

* This column obtained by summation of quantities shown in the preceding columns, and not by measurement.

In Table 14 the Ohio River contributions are added to those of the Mississippi at St. Louis and the total compared with the maximum observed discharge at Columbus, as well as the discharge at Columbus as scaled from rating curves. For the Mississippi River contribution there was used the highest gauge reading which occurred at St. Louis within a period of 2 to 3 days before the flood crested at Columbus.

The sum of the discharges at St. Louis and from the Ohio River does not represent the total discharge at Columbus, because it does not include the run-off from the drainage basins of Meramec, Kaskaskia, and Big Muddy Rivers and a number of smaller streams.

Table 14 shows that the maximum observed discharge at Columbus in 1912 and 1913 was nearly 300 000 sec-ft. more than was measured in 1927, yet the maximum gauge reading in 1927 was 1.6 ft. higher than in 1913. These differences are believed to be due in part to differences in the shape of the flood crests and more particularly to an incomplete levee line in 1912 that induced a slope and velocity that were abnormal at the time the discharge was measured. The discharge at Columbus obtained by summation of the flow of contributing

streams was 1 702 000 sec-ft. in 1912 and 1 614 000 sec-ft. in 1927. On the other hand, if dependence be placed in the rating curve, the 1927 flood exceeded the 1912 flood at Cairo by 67 000 sec-ft. In formulating plans for protection against floods of the future, it will be conservative and on the side of safety to assume a discharge corresponding to a foot on the gauge at or near maximum stages, no greater than that shown by past floods. In the 1927 flood a foot on the gauge at and near the maximum stage corresponds on the average to a flow of 60 000 sec-ft., and, as no flood of the past shows a smaller average discharge, that ratio has been used by the Commission in its present plans and estimates.

TABLE 14.—DISCHARGE, IN CUBIC FEET PER SECOND.

Year.	Mississippi River, at St. Louis, Mo.	Ohio River, from Table 13.	Sum of Columns (2) and (3).	MISSISSIPPI RIVER, AT COLUMBUS, KY.	
				Scaled from rating curves.	Maximum observed.
(1)	(2)	(3)	(4)	(5)	(6)
1890	72 000	1 229 000	1 301 000	1 312 000	
1892	408 000	882 000	1 290 000	1 332 000	1 401 000
1893	601 000	884 000	1 485 000	1 470 000	1 528 000*
1897	283 000	1 240 000	1 523 000	1 464 000	1 462 000
1898	283 000	1 079 000	1 362 000	1 393 000	1 517 000
1903	377 000	1 016 000	1 393 000	1 426 000	1 483 000
1904	383 000	994 000	1 377 000	1 389 000	1 502 000†
1907	429 000	918 000	1 347 000	1 436 000	1 543 000
1912	592 000	1 110 000	1 702 000	1 733 000	2 015 000‡
1913	323 000	1 395 000	1 718 000	1 750 000	2 015 000§
1916	617 000	869 000	1 486 000	1 497 000	1 775 000
1917	285 000	994 000	1 279 000	1 362 000	1 420 000
1920	446 000	869 000	1 315 000	1 422 000	1 527 000
1922	311 000	1 137 000	1 448 000	1 520 000	1 501 000
1927	800 000	814 000	1 614 000	1 800 000	1 728 000‖

* Another measurement at same stage gave 1 434 000 sec-ft.

† Another measurement at about same stage gave 1 447 000 sec-ft.

‡ Measured at 48.5-ft. stage. At the maximum stage the discharge measured 1 854 000 sec-ft.

§ Mean of seven consecutive measurements was 1 959 000 sec-ft.

‖ Cairo gauge was 53.7 ft. at time of observation.

The maximum discharge past the latitude of Arkansas City in April, 1927, has been estimated at 2 662 000 sec-ft. This is 250 000 sec-ft. more than in 1912 and about 200 000 sec-ft. more than in 1916.

The estimated maximum outflow from the latitude of Old River was 2 350 000 sec-ft., about 300 000 sec-ft. less than at Arkansas City. If to this difference the flow of the Yazoo and the Red Rivers is added, a measure of the regulating influence of storage in the lower ends of the Yazoo and Tensas Basins, and in the river itself, is obtained.

Subsequent to the flood of 1927 the Commission was directed by the Secretary of War and Chief of Engineers, U. S. Army, to revise its plans to conform to the new conditions. The Commission has been engaged on that work for the last four months and recently submitted a report to the Chief of Engineers. Although the conclusions and recommendations of the Commission cannot be divulged, it is proposed to outline in a general way the nature of the problem and to refer briefly to some of the methods considered.

As a basis for a new project, it was determined to set up a probable future maximum flow at Cairo. The discharge at Cairo in 1927 was approximately 1 800 000 sec-ft. In determining how much larger a flood should be provided for, consideration was given to the fact that if to the maximum discharge of the Mississippi at St. Louis there was added the maximum discharges of the Wabash at Mt. Carmel, the Ohio at Evansville, the Cumberland at Nashville, and the Tennessee at Florence, the total would aggregate more than 3 000 000 sec-ft. If the maximum discharge of the Ohio and the maximum discharge of the Mississippi at Cairo were added, the total would be about 2 700 000 sec-ft. If to the estimated discharge of the 1927 flood (1 800 000 sec-ft.), there be added the difference in flow, at flood stages, of the Ohio in 1913 (maximum) and in 1927, the total will be 2 400 000 sec-ft.

TABLE 15.

Miles below Cairo, Ill.	Station.	New levee grade.	Existing levee grade.	Increase in levee heights, in feet.
0	Cairo, Ill.	70.4	58.0	12.4
22	Columbus, Ky.	64.7	52.7	12.0
71	New Madrid, Mo.	56.5	47.6	8.9
124	Cottonwood Point, Mo.	53.6	46.0	7.6
175	Fulton, Mo.	54.4	47.0	7.4
227	Memphis, Tenn.	58.4	51.5	6.9
273	Mhoon Landing, Miss.	60.1	50.0	10.1
307	Helena, Ark.	71.0	58.5	12.5
354	Sunflower Landing, Miss.	72.6	56.0	16.6
392	Mouth of White River	78.4	61.9	16.5
437	Arkansas City, Ark.	79.5	60.5	19.0
480	Greenville, Miss.	72.8	55.8	17.0
543	Lake Providence, La.	69.1	53.7	15.4
602	Vicksburg, Miss.	72.0	58.0	14.0
662	St. Joseph, Mo.	68.7	54.0	14.7
706	Natchez, Miss.	72.7	57.5	15.2
773	Red River Landing, La.	69.5	57.5	12.0
807	Bayou Sara, La.	61.8	51.5	10.3
841	Baton Rouge, La.	57.9	48.1	9.8
862	Plaquemine, La.	52.1	43.7	8.4
894	Donaldsonville, La.	46.4	39.0	7.4
911	College Point, La.	40.6	34.4	6.2
965	Carrollton, La.	30.8	25.2	5.6
1 047	Fort Jackson, La.	14.4	11.0	3.4

All these combinations may be classed as "possibilities", but it does not seem probable that rainfall sufficient to produce such coincident floods will ever occur. In its determination of this question the Commission gave full consideration to the views of the local Weather Bureau meteorologist at Cairo and to the views of the Chief of the U. S. Weather Bureau. For the purposes of this paper it is proposed to assume 2 250 000 sec-ft. as the probable maximum flood of the future at Cairo. If the relation between gauge and discharge established in the 1927 flood were maintained, such a flood, confined by levees, would reach a stage of 65 ft. on the Cairo gauge. This assumed discharge is 25% greater than that of 1927, a factor of increase relatively much larger than that assumed in the Miami Conservancy project, when the relative magnitude of the two drainage areas is considered. If protection against such a flood were provided by levees alone, it would require an increase in levee heights as shown in Table 15.

This increase in levee height is based upon a 5-ft. free-board instead of the 3-ft. free-board heretofore used, a modification that was generally recommended by the engineers of local levee districts as a result of their experience with the 1927 flood. These levee engineers were also practically unanimous in recommending that the levee section be strengthened by flattening the river-side slope from 1 on 3 to 1 on 4, by increasing the crown width from 8 to 12 ft., and by flattening the land-side slope so as to contain completely a saturation line of 1 on 7 springing from a point on the river-side slope 2 ft. vertically below the crown. The flattening of the river-side slope was advocated largely as a means of protection against wave wash. The increased free-board and increased width of crown were advocated as increased factors of safety and the flattening of the land-side slope was for the purpose of reducing seepage during long-continued high stages. To increase the levees to such heights and to increase the section as outlined would require an expenditure for main river levees alone of more than $500 000 000.

The Commission in revising its project, considered all the many methods that have been suggested as substitutes for or as adjuncts to a levee system. Many of them are being covered by other papers and to include herein a discussion of them would result in duplication. However, even at the risk of duplication it seems desirable to refer briefly to reservoirs and to spillways and outlets.

Reservoirs

High floods in the Mississippi, as far as records are available, have crested at Cairo between February 22 and May 9, except the flood of 1916. The flood wave at Vicksburg is much longer and flatter than at Cairo and at this point the river is within 4 ft. of its crest for a period varying from 20 to 80 days. The length of the Mississippi River flood crest, the great distance of the head-water reservoirs from the region of the flood, and the difficulty of forecasting these floods, indicate that head-water reservoirs above Cairo to be effective must store water for a period of about 90 days. Data concerning 245 proposed reservoirs were examined and from the storage graphs there was determined the percentage of reservoir capacity which would have been utilized each flood-year.

A first study concerned head-water reservoirs above Cairo and assumed them to be operated solely in the interest of Mississippi River flood reduction. Reservoirs having an aggregate capacity of 71 000 000 acre-ft. were considered, the cost per acre-foot ranging from 50 cents to $112. A storage-cost curve was prepared based on storage at a uniform rate during the storage period, on an efficiency as measured by the quantity of water stored equal to 75% of reservoir capacity, and on a relation between gauge and discharge at Cairo of 1 ft. equals 60 000 sec-ft. From this cost curve it appeared that the cost of reducing flood heights at Cairo would range from $62 000 000 for the first foot to $230 000 000 for the fourth foot, and that a reduction of 4 ft. by means of such reservoirs, equivalent to 240 000 sec-ft., would cost $560 000 000.

A more detailed analysis was then made using actual rates of storage instead of assumed uniform rates and actual efficiency of reservoirs instead

of an assumed average of 75 per cent. Allowance was also made for the effect of storage in the river valleys between the reservoirs and Cairo. This more detailed study was limited to the Ohio Basin and comparison made between the floods of 1913 and 1922. The effect of valley storage was determined approximately by showing graphically the relation between the inflow into the Ohio River from its tributaries and the outflow as measured near its mouth (Figs. 2 and 3). The inflow from the discharge diagrams of tributaries having a total drainage area of 170 100 sq. miles, has been increased by direct ratio to allow for an additional drainage area of 33 800 sq. miles. The composite inflow graph of all the tributaries was synchronized with respect to the outflow graph by taking into consideration the time of travel of water from the mouth of each tributary to the mouth of the Ohio. The difference between the inflow and the outflow curve appears to be a direct measure of the effect of valley storage and the true effect at Cairo of reservoir storage in the Ohio Basin can only be determined by taking into consideration the modifying influence of valley storage. The 1913 flood in the Ohio Valley was characterized by a sharp peak whereas that of 1922 was a long low flood. In the latter, valley storage was a much less important factor than in the former.

The cost and effect of reservoirs was analyzed also on the assumption of joint use, that is, a combination of Mississippi River flood control with local flood control, the development of power, and improvement for navigation, or by a combination of three or more uses. A great many such combinations are possible and although the several uses are more or less conflicting, it may be said generally that such joint use will result in some reduction of cost per acre-foot chargeable to Mississippi River flood control.

A group of the most desirable reservoirs above Cairo, as determined by the ratio of benefits to costs, was then selected for further study. This group had a capacity of about 38 000 000 acre-ft., the average gross cost being between \$10 and \$11 per acre-ft., and the net cost chargeable to Mississippi River flood control between \$7 and \$8 per acre-ft. To reduce flood heights at Cairo by this selected group would require a gross expenditure for the first foot of reduction of \$104 000 000, or a net expenditure chargeable to the Mississippi River of \$40 000 000, assuming that the other interests benefited could be induced to pay the difference. The second foot of reduction would cost \$160 000 000, of which it is estimated that \$17 000 000 could equitably be charged to other interests. The average effect of this group in reducing gauge heights at Cairo during past floods would have been 2.65 ft., and had the entire group been built the average net cost under the assumptions made would be \$115 000 000 per ft. of reduction on the Cairo gauge, or nearly \$10 000 000 per in. of reduction. Expressing the reduction in terms of discharge, the cost would average \$190 000 000 per 100 000 sec-ft. for a maximum reduction of 159 000 sec-ft.

Suggested reservoir sites on the St. Francis, White, and Arkansas Basins were also considered. These reservoirs are generally located in areas of heavy rainfall, which are important contributors to floods in the Lower Mississippi, and the proposed reservoirs are of such size that their capacity could be

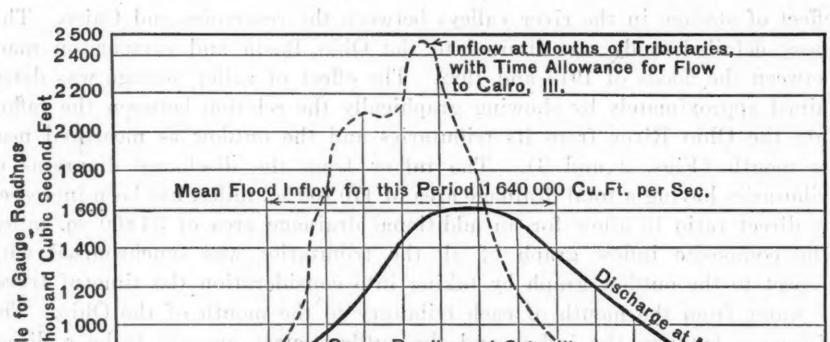

FIG. 2.—INFLOW AND OUTFLOW OF OHIO RIVER DURING 1913 FLOOD.

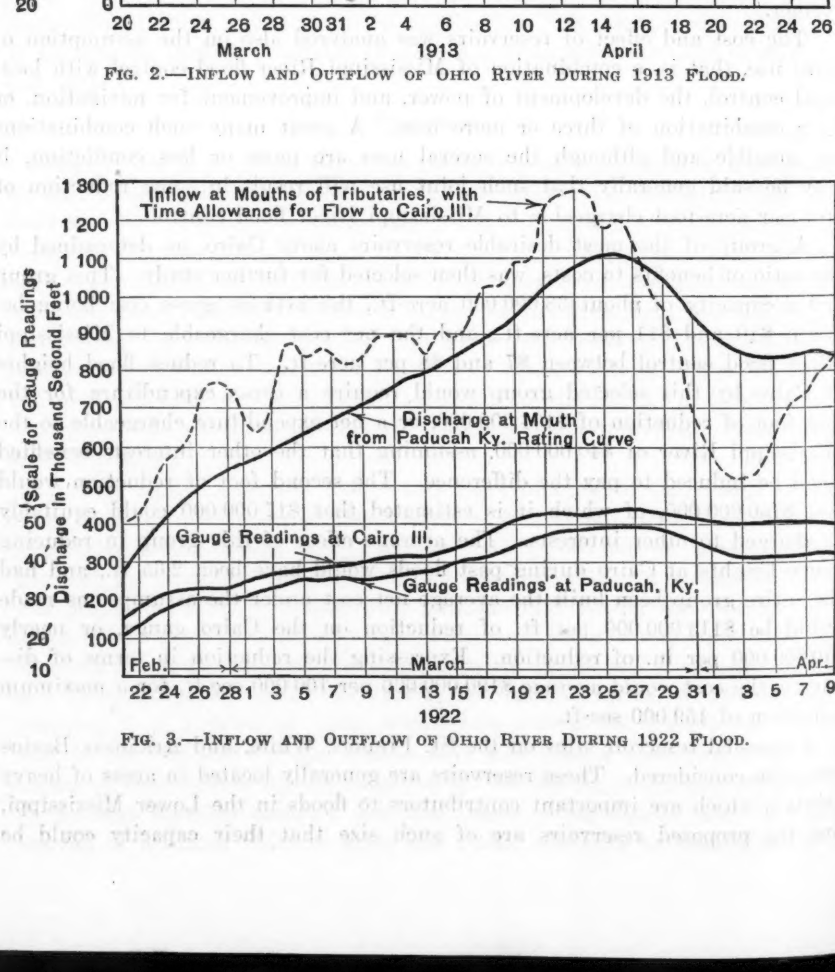

FIG. 3.—INFLOW AND OUTFLOW OF OHIO RIVER DURING 1922 FLOOD.

effectively utilized during most Mississippi River floods. These reservoirs have a gross capacity of 25 000 000 acre-ft. and are estimated to cost $215 000 000, or $8.50 per acre-ft. This cost may be reduced under a joint-use plan; furthermore, by reason of their proximity to the delta they should be more valuable than an equal amount of storage 1 000 miles above Cairo.

Studies were also made of suggested reservoirs in the St. Francis, Yazoo, and Tensas Basins. About thirty years ago the late James A. Seddon, M. Am. Soc. C. E., proposed a series of reservoirs in the St. Francis Basin as a means of regulating Mississippi floods. A similar plan, modified to conform to developments that have taken place in the basin since that date, was considered but the cost of storage was found to be approximately $27 per acre-ft. Estimates were also prepared of the cost of reservoirs at the lower ends of the basins. Such reservoirs would substitute controlled storage for the uncontrolled storage which now exists at the lower ends of these basins, and the cost of such reservoirs must be balanced not against their value in reducing flood heights, but against the difference in value of controlled and uncontrolled storage.

Outlets

The reduction of flood heights in the Lower Mississippi River by the diversion of part of the flood waters has been the subject of discussion for many years. In presenting the matter, the following terms will be used with the meanings stated: An outlet is a natural channel serving to extract water from the river at all stages and conduct it directly or through collateral waterways to the Gulf. A diversion channel is an artificial one serving to divert part of the main river flow to the river at a lower point or by independent collateral channels to the Gulf, the flow into it being wholly or partly under control. A spillway is a structure designed to afford regulated entrance of flood waters into a diversion channel.

At present, there is an outlet at Old River 202 miles above New Orleans. Old River is a link connecting the Mississippi with the Red-Atchafalaya System. It serves as a relief to Mississippi flood water except when higher flood stages in the Red-Atchafalaya System cause a reversal of flow. To a limited extent, it is used for light-draft navigation, suitable depths being maintained by dredging.

Although described as an outlet its size and the flow through it at low stages are controlled to some extent by several mattress sills near Simmesport, La. This route to the Gulf is 150 miles shorter than that by way of the main river. It is now leveed in the upper 40 miles and its capacity at flood stage is about 400 000 sec-ft. It has been suggested that the levees on one or both banks be set back so as to create a floodway with a capacity of 900 000 to 1 000 000 sec-ft. To secure such a flow without excavation would require a width between levees of 6 to 8 miles and entail the acquisition of extensive flowage rights through what is known as the "sugar bowl" of Louisiana.

Whether such a floodway should be controlled by a spillway so as to confine in the main river all or part of the flow below a bank-full stage is an important question. Such a spillway would interrupt navigation between the Mississippi and the diversion channel unless a lock were also provided. On the other hand unless controlled, so large a diversion may cause silting in the main channel to such an extent as to interfere with navigation at low stages and to diminish its flood-carrying capacity at high stages.

A diversion channel through the Tensas Basin with capacity ranging from 600 000 to 750 000 sec-ft. was also considered. Such a channel would leave the main river at or near the mouth of the Arkansas and extend to the upper end of the proposed floodway through the Atchafalaya Basin. Study was also made of a diversion channel through the St. Francis Basin, with capacity ranging from 300 000 to 500 000 sec-ft. Unless such a channel were extended to the White River and thus connected with the Tensas Basin diversion, it would not permanently divert water from the main channel.

In addition to these three major diversion plans, consideration was given to the construction of several short diversions with capacity of 250 000 to 300 000 sec-ft. above and below New Orleans, primarily for the protection of that city.

Diversion of part of the flood flow through the St. Francis Basin will apparently cost more than to confine the entire flow between levees on the main river. Diversions through the Tensas and Atchafalaya Basins are far more promising from the standpoint of first cost. Even if cheaper than the plan which would confine the entire flow between levees, the creation of such secondary channels for the discharge of part of the flood flow can be justified only if the opponents of diversion channels are mistaken in their belief that they will not permanently lower flood heights.

Opponents of outlets use as an illustration the changes in the relation of stage to discharge at Red River Landing, a short distance below the Atchafalaya Outlet. The records of the Mississippi River Commission show for the flood of 1882 a measured discharge of slightly more than 1 500 000 sec-ft. at a stage of 48.5 ft. In 1912, practically the same discharge was measured at a stage of 53.2 ft. and, in 1927, at a stage of 57.5 ft. The increase in flood height of 9 ft. between 1882 and 1927 without increase in volume of discharge is attributed by the opponents of outlets to deterioration in the carrying capacity of the channel of the main river due to the effect of the Old River Outlet. (The difference between 1882 and 1912 may be entirely accounted for by the difference in slope in the two cases caused by crevasse effect below.)

Evidence obtained from a study of crevasses supports the view that an outlet or secondary channel that diverts water at all stages is detrimental to the main stream and that the larger it is in proportion to the main stream, the greater will be the deteriorating effect upon the main channel. Such a diversion tends to cause a raising of the bed below the outlet, thereby reducing the depth available for navigation at low stages and reducing the flood-carrying capacity of the stream at high stages.

Opponents of controlled diversions, even if the diversion be limited to over-bank flow, argue that if water be taken out of the river at any point the level will be lowered at that point and the slope of the river increased above the point of diversion and reduced below that point; that the increased velocity above will increase the silt-carrying capacity of the river and that it will approach the spillway with a greater load of silt than it would have had under natural conditions; that the reduced slope and reduced velocity below the spillway will result in sedimentation and reduction in the cross-section of the river; that this will continue until the river is again able to carry off its portion of the sedimentary matter; that the necessary increase in velocity can only be obtained by increased head; and that increased head means an increased flood level.

This belief that the Mississippi River is so charged with sediment that the reduction in slope and velocity below a controlled diversion channel will cause deposits, is not borne out by the findings of Humphrey and Abbot. Furthermore, if the diversion is so built that it will not come into action until after the river reaches a bank-full stage (approximately 1 000 000 sec-ft.) and so controlled that the flow through it can be reduced as soon as the crest passes, the changes in slope and velocity can be kept within such narrow limits that deposits should not occur. Even if some sedimentation does take place, it should be completely removed later by the river itself, if the spillway is closed as soon as the discharge of the river falls within the capacity of the main channel.

Conclusion

Protection against an assumed probable maximum flood at Cairo of 2 250 000 sec-ft. can be provided by enlarging and strengthening the existing levees, but no matter how well the levees are built nor how well they are protected by revetment of the banks, an 1 800-mile levee line of the height necessary to contain such a flood would carry with it a menace to the people living behind it that should be avoided if practicable.

Head-water reservoirs above Cairo as a means of reducing levee heights will apparently cost very much more than a series of diversion channels or floodways through the delta basins.

Some of the proposed diversions will apparently justify themselves from an economic standpoint in that they will cost less than corresponding increase in levee heights; others will cost more, but may be justified by reason of the greater safety that they will afford the inhabitants.

Such secondary channels are not substitutes for levees on the main river. Even with all the proposed diversion channels built, the existing levees will have to be maintained and even increased in height in order to provide protection against the assumed flood of 2 250 000 sec-ft., at Cairo.

Whatever may be assumed as the probable maximum flood of the future, an even greater flood may some day occur, sufficient to overtop the levee line. To provide for such a contingency, it has been proposed to construct along

the crown of the levee a series of over-bank spillways that would be brought into play only when overtopping was imminent. Such structures would provide the means of substituting a minor disaster for a major one and permit prompt recovery from the effects of such a super-flood.

In the writer's opinion, the most dependable, as well as the cheapest, method of protecting the delta basin of the Mississippi River against a probable future flood lies in a system of main river levees, supplemented by a series of controlled diversion channels through which excess flood waters can be carried to the Gulf.

As an integral part of such a flood-control plan, the main river should be stabilized by revetting the caving banks. While such stabilization is regarded as an essential part of the flood-control project, it will also greatly benefit navigation at low stages.

FOREST COVER AS A FACTOR IN FLOOD CONTROL

By E. F. McCarthy,* Esq.

In an address made before the Mississippi Flood Control Conference at Chicago, Ill., early in June, 1927, United States Forester W. B. Greeley took the position that while the main reliance for handling large flood discharges must be placed upon engineering structures, forests have a definite part in flood control, together with other forms of land use which check erosion and favorably influence natural storage conditions. This attitude is the rational one, and engineers facing the problem of flood control in the Mississippi will doubtless feel that any agency which will assist in lowering the flood crests and in retaining the flood water within predetermined bounds should be used if economically feasible.

The relation of forest to stream flow is a subject which has received extensive discussion. Several good bibliographies have been collected. One is incorporated in the Final Report of the National Waterways Commission,† and a more recent one is contained in a report by the California State Board of Forestry.‡ No attempt has been made to repeat references in either of these publications. The Forestry Committee on the Relation of Forests and Waters submitted the following statement to the Fifth National Conservation Congress:§

"In the mountains, the forests break the violence of rain, retard the melting of snow, increase the absorptive capacity of the soil cover, prevent erosion, and check surface run-off in general, thus increasing the underground seepage and so tend to maintain a steady flow of water in streams."

Zon† has summarized the effect of forests on stream flow, as follows:

"Among the factors, such as climate and character of the soil, which affect the storage capacity of a water-shed, and therefore the regularity of stream flow, the forest plays an important part, especially on impermeable soils. The mean low stages as well as the moderately high stages in the rivers depend upon the extent of forest cover on the water-sheds. The forest tends to equalize the flow throughout the year by making the low stages higher and the high stages lower.

"Floods which are produced by exceptional meteorological conditions cannot be prevented by forests, but without their mitigating influence the floods are more severe and destructive."

A study has been conducted by the U. S. Forest Service during the summer of 1927 to determine the relation of the forests to floods in the Mississippi.

* Director, Central States Forest Experimental Station, Forest Service, U. S. Dept. of Agriculture, Columbus, Ohio.

† "Forest and Water in the Light of Scientific Investigation," by Raphael Zon, Final Rept., National Waterways Comm., Appendix V.

‡ "Erosion and Flood Problems in California," by E. N. Munns (Rept. by California State Board of Forestry to the Legislature (1921) on Senate Concurrent Resolution No. 27).

§ "The Relation of Forests and Water," by Raphael Zon and others, Sub-Committee on Forest Investigations.

The collection of information relative to the Ohio drainage came under the supervision of the Central States Forest Experiment Station, so that direct reference will be made to this tributary.

The Ohio Basin, which contains approximately 203 230 sq. miles, once practically all forested, now has an estimated forest cover of 68 220 sq. miles. About 33 120 sq. miles of this land is woodland incorporated in the farms, of which about one-third is pastured. These statements have been qualified carefully because exact information in regard to the actual forest cover and its condition has never been collected for this region. In only a few instances, notably in Illinois, has an effort been made actually to map the existing woodland. The best general statistics are those included in the Census of Agriculture for 1925, and even this report does not adequately classify land lying outside the farms so that an accurate judgment can be made as to the amounts of forested and non-forested wild land. The foregoing statement of woodland acreage is based on the Census report, modified by the writer's judgment of the forested acreage of wild land. Of course, all available information which could be procured from State and individual sources has been used. From this summary the Ohio Valley may be considered as one-third forested.

The present trend, especially in the mountainous and hilly sections, is to abandon farm land rather than to extend its present acreage by further clearing. This tendency is so obvious as to attract even casual observation. The causes are chiefly three: (1) The inability of production on rough land to compete with that on farms more favorably situated as to soil, topography, and market; (2) the severe erosion and destruction of the steeper fields; and (3) industrial development and improvement of road communications which is draining the forested area of its labor element.

The forest of the Ohio Valley is largely made up of broad-leafed species. The exceptions to this are a comparatively limited acreage of spruce and fir forest in the higher plateau of West Virginia and the Southern Appalachian Mountains, and the mixture of scattered pine and hemlock found in some places with the broad-leafed species. The more exposed ridges also may support small patches of pure yellow pine. From this statement it is obvious that this entire forested area each year receives a carpet of leaf litter, chiefly from hardwoods, which are left largely defoliated during the winter season.

The forest of the Ohio Valley is now very unsatisfactorily stocked. Even in its virgin condition there were large spaces in the forest crown cover. Cutting, fire, and grazing have made further serious inroads on the virgin stand. The older timber now left is only a remnant of the original stand. These influences have, in turn, increased the general undergrowth of rhododendron, laurel, and other shrubs. Significant, in so far as flood prevention is concerned, is the thinness of the stand which now occupies the forested land of this valley. Only in the few places which have received adequate protection from fire for a considerable period of years is there a dense stand of timber with a well-covered forest floor. In the past the rougher mountain and plateau lands have been frequently burned over. In 1927, 211 424 acres

of land were reported burned in Ohio, Indiana, Kentucky, Tennessee, and West Virginia. This represents about 0.7% of the total wooded acreage of these States. The foresters of the Ohio Valley States recognize the necessity for better protection from fire and improved management of timber lands, both for increasing the protective value to their water-sheds and for the yield of timber.

The relation of the forest to run-off is most closely established through its influence on erosion, since it is through the washing away of the finer soil particles that channels are established for the flow of water. The first place to seek control of a flood is at its source where every small impediment is most effective in delaying the union of raindrops into rivulets and these, in turn, into streams strong enough to sweep clear a direct channel. The best that can be done at the source is to secure the most effective percolation of water from the surface to the water-table.

The fact is well established that the forest, through its littered floor and porous soil, does cause impediment to the flow of water on the ground surface and does increase the water storage, thereby maintaining the flow of springs in dry seasons. A large volume of evidence has been accumulated in support of this contention. Several striking examples are cited.

In 1922, Charles G. Adsit, M. Am. Soc. C. E., Vice-President and Executive Engineer of the Georgia Railway and Power Company, stated:

"At our Morgan Falls Plant on the Chattahoochee River, which is below large cultivated areas, the reservior in 1904 covered 750 acres. After ten years' service, there was no reservoir capacity worth mentioning left, it having all silted shut, leaving only a channel through the center of the reservoir for the river flow. On the other hand, our Mathis Reservoir, above which is only forested area, after ten years' service, shows little or no sign of silt whatsoever."

The following statement is made in a recent letter by Edgar P. Kable, General Manager, The York Water Company, York, Pa.:

"Our observations of the effect of forest planting on our water supply in dry times has been that it has increased the dry weather run-off very materially.

"We have a 700-acre tract planted with 680 000 evergreen trees and although we have no gauge at present, nor did we have prior to the building of the dam, yet this fact is noticeable.

"The quality of the water in the stream has also materially improved as there is very little erosion from the banks, and, as an example, when there was a great downpour of rain * * *, the water in this dam remained clear, whereas some of the neighboring streams became very turbid."

The litter-covered forest floor is generally recognized as the best protection against erosion of soil. Through the action of the litter in preventing the packing of soil by rains, the forest soil remains in a loosened condition throughout the season. The soil is loosened by the action of frost, as well as rodents, and other animal life, and is especially porous in the early spring months at the time when the precipitation or accumulation of snow water is greatest. A mere comparison of the forest soil made porous by these agents and by the penetration of tree roots, with the packed soil of adjacent fields,

will convince any one of the value of this cover in retarding run-off. The action of the litter in delaying run-off is most pronounced in the first few rods of movement of the water while it is accumulating in the natural drainage channels.

The terrific loss of soil through erosion is not generally understood, and an excerpt from *Research Bulletin 63* of the Missouri Agricultural Experiment Station* is inserted to bring out this point. Experiments were conducted for a period of six years on carefully controlled plots with an average gradient of 3.68 per cent. In summarizing the results, the writers state:

"If farm land should erode as rapidly as the land in these experiments the surface 7-in. layer would be removed at the following rates: uncultivated land in 29 years; plowed 4 in. deep in 24 years; plowed 8 in. deep in 28 years; planted to corn annually, in 56 years; wheat, annually, in 150 years; rotation, corn, wheat, and clover in 437 years; bluegrass sod in 3 547 years.

"The chemical analyses showed that the amounts of nitrogen, phosphorus, calcium, and sulphur in the eroded material from corn or wheat land may equal or exceed the amounts taken off in the crops. There were only small amounts of nitrogen, as nitrates, lost in the run-off water."

This Missouri experiment brings out the great difference in loss of soil from bare or cultivated land and that protected by bluegrass sod. While it does not afford evidence of the protective value of the forest, it does indicate the loss which might be expected, even on comparatively level land, if a sod cover should fail and leave the land bare.

The most striking example of complete removal of soil in a forest region is afforded by the area damaged by smelter fumes near Ducktown, Tenn. Here, each few feet of surface has developed its water channels and tremendous quantities of the surface soil have been washed into the tributaries of the Hiwassee River. The significant thing in connection with the Ducktown area is that the soil accumulated during ages, under the protection of a forest cover, was eroded in a few years when exposed by the damage from smelter fumes.

The dry weight of hardwood litter annually deposited on the forest floor has been estimated at about 2 tons per acre. This will vary somewhat according to the species and the density of the stand. The rapidity of disintegration also varies, as influenced by wind and weathering. For use in this paper, determinations were made on plantations averaging about 17 years of age, located at the Ohio Agricultural Experiment Station, at Wooster, Ohio. The work was done in the week ending October 8, 1927, before the leaf crop had fallen. The litter on sample areas was weighed and its moisture content determined. The dry weight of litter per acre on these plantations varied from 1 827 lb. to 17 545 lb. and is able to absorb 1½ to 2½ times its dry weight in water. It is notable that the litter underneath the soft woods, pines, spruce, and Douglas fir, exceeds that of the hardwoods at this time of year.

The relative influence of forested and non-forested areas on run-off and erosion has been specifically studied at several places in recent years. The most notable of these, perhaps, is the work being done by the U. S. Forest

* "Erosion and Surface Run-Off Under Different Soil Conditions," by L. F. Duley and M. F. Miller.

Service in co-operation with the U. S. Weather Bureau at the Wagon Wheel Gap Station, in Colorado. Here two water-sheds as nearly similar as could be found were subjected to measurements for a period of years. One of them was deforested and the measurements were then continued for a further period of years. A preliminary report of this work has been made,* and the final report is now in preparation. Experience gained in the Wagon Wheel Gap experiment shows the extreme difficulty of securing adequate values through the studying of paired water-sheds.

A less extensive study of run-off has been made by C. E. Ramser,† M. Am. Soc. C. E. While attempting to determine the drainage requirements for road purposes, he found the forest an important factor in controlling the rate of run-off. He says of his results, "these values show quite conclusively that timber has a decided influence in reducing the rate of run-off from a water-shed". His work is further quoted in the Appendix.

A recent paper by Mr. H. Burger,‡ reports a study in Switzerland, where a comparison was made of forested and non-forested water-sheds. He found that the crest of the flood was not only delayed by the forest cover, but that the total run-off per square kilometer was also reduced. In both these instances neither water-shed was completely covered by forest, but a sufficient difference existed to bring out these facts.

The most conclusive study made thus far is that on two water-sheds in the White Mountains.§ In comparing two water-sheds, one largely forested and one deforested and open, the following conclusions have been drawn: At the end of each period of observation there was more snow on the forested than on the deforested basin; the loss of water from snow storage was heavier on the deforested basin; the rate of run-off from the forested basin was less than two-thirds that from the deforested basin; the maximum flood flow from the forested basin never exceeded 71% of that on the deforested basin; and, lastly, considering the precipitation over the entire period, the forested basin added largely to the ground-water storage, whereas the deforested basin drew in large amounts from that storage.

Mr. I. T. Bode,‖ in describing the results of his work in Iowa, claims an increase of water storage in forested lands as compared with the open prairie land adjoining.

In considering the effect of forest cover on the catchment of precipitation, the influence of the trees themselves in intercepting both rainfall and snow-fall must be considered. This influence is particularly important in light rains when a large proportion of the precipitation may be intercepted by the tree crowns and re-evaporated into the air without having reached the forest soil.

* Supplement No. 17, *Monthly Weather Review.*

† "Run-Off from Small Agricultural Areas," *Journal of Agricultural Research,* Vol. 34, No. 9.

‡ *Swiss Forestry Journal,* March, 1927.

§ "A Preliminary Statement of the White Mountains of New Hampshire," by George N. Smith, U. S. Geological Survey Rept., No. 13.

‖ "The Relation of the Smaller Forest Areas in Non-Forested Regions to Evaporation and Movement of Soil Water," Iowa Academy of Science, Vol. XXVII, 1920, pp. 137–157.

Robert E. Horton, M. Am. Soc. C. E., has discussed this subject,* and has also investigated the loss of water by transpiration from trees.† He indicates that from 0.02 to 0.07 in. of rainfall per shower may be intercepted, and that the total loss for the summer season may be as high as 40% of the total precipitation.

Mr. H. S. Graves‡ has reported loss through interception in forests as varying from 20 to 60% of precipitation. While no very exact information exists, and the interception is admittedly less in regions of heavy rains, the loss through interception of the crowns of trees must be considered, together with transpiration and ground storage, as a factor in flood control.

It is evident that there is still a large field for research in determining the exact relation of the forest to the retention of flood waters. In spite of this fact, the evidence collected thus far is of a positive nature and indicates a material and beneficial influence.

The rapidly increasing number of storage reservoirs for power purposes and the increasing value of forest-covered water-sheds as sources of potable water for neighboring cities, are additional reasons for the maintenance of good forest conditions on the upper reaches of mountain streams. Not only should the abandoned fields of the Appalachian Mountains and Cumberland Plateau tributary to the Ohio drainage be forested, but many areas of cleared land in the hill sections of Illinois, Indiana, and Ohio, can be restored to forest cover both with benefit to the flood situation and profit to the owners.

In summarizing the influence of the forest on flood control, the outstanding service which it accomplishes is the protection of the rougher land from erosion. This, in turn, has a direct influence through the control of silting on practically all the engineering work undertaken.

The forest is a natural storage for water, because it delays run-off, intercepts rainfall, and retains the fine soil and humus in place on the steeper slopes.

APPENDIX

China has long been held up as a terrible example of forest destruction. Mr. W. C. Lowdermilk§ has recently prepared a preliminary report on the problem of forest conservation in Shansi, China. The following abstract is taken from this report:

"The Taiyuan plain was suffering from a long drought when we reached Taiyuanfu. The crops were dying for want of rain, even the wild grass had not started (July 3) and a famine was feared.

"The rain clouds broke shortly after our arrival. The moisture which had been accumulating in the atmosphere, superheated by the roasting hills, came down in torrential volumes. The streams were suddenly overcharged with raging waters, heavily laden with mud and silt. The water supply so sorely needed for agricultural crops quickly ran off the steep barren slopes and

* "Rainfall Interception," *Monthly Weather Review*, Vol. 47, No. 9, pp. 603–623.

† "Transpiration by Forest Trees," *Monthly Weather Review*, Vol. 51, No. 11, pp. 571–581.

‡ *Monthly Weather Review*, Vol. 42, December, 1914, p. 671.

§ "The Problem of Forest Conservation in Shansi, China." (In mss.)

brought floods and destruction in the place of the desired benefits of rain after drought. Despite the efforts made to catch the flood waters for irrigation, most of them soon passed by and were gone. Dry weather in the fall months again made the conditions of the people serious and threatened them with famine. Being deprived of their soil and forest cover, the mountains no longer serve as a storehouse of water to be made available, when most sorely needed, to the fields in the plains.

"Our party was able to study these forces at work and to follow the processes of the denudation of mountain slopes. Far back at the foot of the famous Lu Yah Shan near the Great Wall was found a small remnant of what was once a great forest cover. Now only a few score of square li of the original forest are left. Yet here may be found the beneficent influences of a forest cover. From out this natural forest area, consisting of trees and a deep porous soil of decaying vegetable matter, flows now a beautiful perennial stream. It was scarcely affected by the heavy rains of the summer [1924], whereas the stream beds draining the barren slopes became raging torrents after an hour of rainfall. But now this last remnant of true and natural forest is being attacked, for not less than 500 wood cutters began in the summer of 1924 to clear the tract.

"The cutting of trees is not the cause of forest destruction in the mountains of Shansi; it is, on the contrary, the digging up of the soil on the mountain slopes for agricultural crops that begins the process of destroying the soil cover."

The following is an abstract from the paper by Mr. H. Burger* previously mentioned:

In two small water-sheds in the Canton of Berne, Switzerland, measurements have been taken which indicate very definitely the effect of the forest cover on run-off. These two water-sheds are similar in altitude, exposure, slope, and character of soil. They are 3 km. distant from each other. One (the Sperbelgraben) is entirely wooded, the other (the Rappengraben) is only 35% wooded. The entirely wooded basin has an area of 56 hectares, the other, 70 hectares.

On June 22, 1926, at 5:00 P. M., a violent storm took place, during the course of which 28 mm. of rain fell on the wooded basin and 29 mm. on the other. The maximum flow from the partly forested basin was 1 079 liters per sec. for each 100 hectares and the crest occurred ½ hour after the beginning of the rain. The maximum flow from the completely forested basin was only 240 liters per sec. and the crest occurred about 5 hours after the storm began.

The total run-off per square kilometer during the period from the beginning of the storm until June 23 at 6:00 A. M., was 7 696 cu. m. on the partly forested basin and only 4 720 cu. m. on the completely forested basin. By subtracting the amount of normal flow, observed just before the storm in each case, the run-off due entirely to the storm is obtained. This is 6 184 cu. m. for the partly forested basin and 2 970 cu. m. for the completely forested basin. On the partly forested basin, 21% of the storm-water ran off; on the completely forested area only 10 per cent.

The purpose of the experiments made by C. E. Ramser, M. Am. Soc. C. E., was to determine the rates of run-off from small agricultural areas. They consisted in making rainfall and run-off measurements on six water-sheds ranging in area from 1¼ acres to 112 acres. The following quotation from Mr. Ramser's report† indicates the effect of timber upon rate of run-off:

* *Swiss Forestry Journal*, March, 1927.

† "Run-Off from Small Agricultural Areas," *Journal of Agricultural Research*, Vol. 34, No. 9, pp. 797–823.

"The effect of timber upon the rate of run-off is shown by a comparison of the results obtained for Water-Sheds Nos. 1 and 4, where the timbered areas were 14 and 38.9% of the respective water-sheds. In Table 8 it is seen that the run-off coefficients for Water-Shed No. 1, for all rains except that of July 18, range from 0.33 to 0.49, and for Water-Shed No. 4, from 0.22 to 0.35. These values show quite conclusively that timber has a decided influence in reducing the rate of run-off from a water-shed. However, the results obtained for the rain of July 18—for which run-off coefficients of 0.51 and 0.46 for Water-Sheds Nos. 1 and 4, respectively, were obtained—tend to show that the effect of timber in reducing run-off is slight when the maximum rate of run-

TABLE 8.—Effect of Timber on Run-Off Coefficients for Water-Sheds Nos. 1 and 4; Water-Shed No. 1, 14.0 Per Cent. in Timber; Water-Shed No. 4, 38.9 Per Cent. in Timber.

Date of rain, 1918.	Average Rate of Rainfall During Time of Concentration, in Inches per Hour.		Coefficient of Run-Off (Ratio of Maximum Rate of Run-Off of Average Rate of Rainfall).		Rainfall Prior to Period Taken as Time of Concentration, in Inches.	
	Water-shed No. 1.	Water-shed No. 4.	Water-shed No. 1.	Water-shed No. 4.	Water-shed No. 1.	Water-shed No. 4.
February 19	2.40	1.89	0.40	0.22	0.39	0.37
April 16	3.84	2.92	0.33	0.29	1.21	1.19
April 28	4.44	3.77	0.41	0.25	0.09	0.09
May 7	3.60	3.51	0.38	0.28	0.18	0.18
May 12	1.92	1.89	0.41	0.30	0.10	0.10
May 23	3.12	2.83	0.49	0.35	0.38	0.38
June 1	3.24	3.00	0.36	0.28	0.46	0.38
June 6, 11:00 P. M.	2.28	2.14	0.42	0.28	0.72	0.69
July 18	4.20	3.51	0.51	0.46	0.76	0.71

off occurs after considerable rain already has fallen. This is to be explained by the fact that interception and percolation on timbered areas are much greater at the beginning of a rain than later, so that an increasingly greater proportion of the rainfall runs off as the rain continues. The falling rain is intercepted by the trees, and the cover of leaves on the ground, until saturated, absorbs a large portion of the rainfall. On April 28 the average rates of rainfall were greater than on July 18, yet the rates and coefficients of run-off were smaller, being 0.41 and 0.25 as compared with 0.51 and 0.46 for the rains of April 28 and July 18, respectively. However, the rain that fell prior to the time of concentration on April 28 was 0.09 in. for both water-sheds, while on July 18, it was 0.76 in. for Water-Shed No. 1 and 0.71 in. for Water-Shed No. 4."

RESERVOIRS FOR MISSISSIPPI VALLEY FLOOD PROTECTION

By William Kelly,* M. Am. Soc. C. E.

For many years there has been an insistent demand for reservoirs to regulate stream flow throughout the United States in the interest of flood protection, navigation, irrigation, drainage, power development, and various other works connected with water resources. In fact, the idea that reservoirs will cure most of the evils from which the country is suffering has been widely accepted as more or less axiomatic in spite of the fact that such projects as have been studied in detail have presented so many difficulties that few of them have been found economically feasible. For five years, as Chief Engineer of the Federal Water Power Commission, the writer spent most of his time studying reservoir possibilities throughout the country to determine whether proposed power developments would conform to a general plan insuring maximum economic use of the water resources for all purposes, including irrigation, flood protection, navigation, etc. As a result of this experience the writer, while not accepting reservoirs as cure-alls, believed that they might form a valuable part of a complete Mississippi flood project.

So far as the Mississippi Valley is concerned, the Mississippi River Commission has often considered the subject of reservoirs, but has never found a possibility of obtaining results that would justify a recommendation for diverting any of its limited funds for this purpose. Many think that the reservoir plan for Mississippi flood relief has not received the consideration its merits. Be that as it may, General Jadwin, the present Chief of Engineers, has determined to have the subject studied as thoroughly as all other phases of the flood problem and if there are any reservoirs that can profitably be developed for Mississippi flood protection, they will undoubtedly be included in the project recommended.

It should be noted that the policy of giving Federal aid for flood protection is a quite recent development. Federal jurisdiction over streams is conferred by the Commerce Clause of the Constitution. Twenty-five years ago such jurisdiction and responsibility was strictly limited to improvements for navigation. There has been no change in the Constitution in this respect, but gradually popular demand has caused Congress to extend Federal assistance for flood protection on the principal navigable streams to a point where its relation to navigation improvements is almost lost to sight. Perhaps if Congress had not so acted the public demand would have been strong enough to amend the Constitution.

The great Mississippi flood of 1927 has renewed the demand for further Federal activity, and in recognition of this demand and of the policies developed by Congress, the Administration has directed studies to be made of

* Col., Corps of Engrs., U. S. A.; Vice-Pres., Buffalo, Niagara, & Eastern Power Corporation, Buffalo, N. Y.

the Mississippi flood problem on a much more comprehensive basis than ever before. Early in June, 1927, a board of seven Engineer Officers was appointed to study the reservoir problem. The Board was given broad instructions to consider the question from every angle. It was authorized to call upon District Engineer Officers of the Army for any assistance needed, and to get data from every source where they could be found. The only limitation placed on the Board was that its report should be completed in time to be considered with the reports of boards studying other phases of the flood problem, so that any reservoirs found to be feasible, could be included in a complete report on Mississippi Valley flood protection to be submitted to Congress in December, 1927.

The Reservoir Board immediately laid out a program of studies along the following lines:

1.—To determine the effect on Mississippi floods of the larger reservoir systems already built in the Mississippi Drainage Basin as a practicable indication of what can be done.

2.—To determine reservoir capacities, costs, and effect on Mississippi floods of all practicable reservoirs on the tributaries of the Mississippi, both when operated primarily in the interest of Mississippi floods and when operated to get the maximum economic results, considering all uses of the reservoirs.

3.—To determine reservoir possibilities in the valley of the Mississippi and its main tributaries.

The task set the Board is a tremendous one to complete in the time available; but thanks to the excellent organization placed at its disposition and to the data available from previous studies, it is believed that the results obtained will be sufficiently complete and reliable to determine what can be done with reservoirs to ameliorate floods in the Mississippi, and what such amelioration will cost. Unfortunately, the final results of the Board's studies are not as yet available, so that this paper is no more than the writer's digest of the data now at hand. Some of the figures will doubtless be modified when the final report is made.

In order to determine the effect of reservoirs on Mississippi flood flows, it is necessary to establish some rule for reservoir operation. Practically all large Mississippi floods have occurred during the 90 days from February 15 to May 15. While there have been a few fair-sized floods both before and afterward, it is believed that this period fairly covers the time within which disastrous floods will occur.

The time consumed by water released from reservoirs on tributaries in traveling to the Mississippi varies from 3 days to 5 weeks, with an average of more than 2 weeks. Large Mississippi floods are produced by widespread heavy rainfall occurring throughout the valley when the river is already running full. The Board has been unable to find any means of predicting the time of occurrence of a large flood in the Mississippi as much as two weeks in advance of its occurrence. Hence, on the average the best dependable results from reservoirs will be obtained by filling them as uniformly as the stream flow will permit so as to hold water back from the Mississippi during the 90 days

of probable flood, February 15 to May 15. In its studies the Board has adopted such a procedure as the rule for reservoir operation. It is possible that for some of the reservoirs a more efficient rule of regulating might be found as a result of experience in their operation, but it is not likely that the increased efficiency would more than offset the probable error due to determining the dependable effect from only five floods, as the Board has been forced to do. A brief résumé of the studies on existing reservoir systems will be given.

Miami Conservancy District

These reservoirs are designed to protect the cities on the Miami River from disastrous local floods. Their action, which is automatic, is effective only during floods of considerable magnitude. It consists of storing all the flood flow in excess of the capacity of the valley below the reservoirs. As soon as the natural flow has fallen below the danger point, this stored water is discharged at a rapid rate. The effect on moderate local floods is a minor one and small floods are entirely unaffected. The costs and capacities of the reservoirs are as given in Table 16.

TABLE 16.—Costs and Capacities of Miami Conservancy District Reservoirs.

Reservoir.	Capacity, in acre-feet.	Cost.	Cost, per acre-foot.
Taylorsville	186 000	$4 170 000	$22.40
Lockington	70 000	2 020 000	28.90
Huffman	167 000	3 680 000	21.70
Englewood	312 000	4 050 000	13.00
Germantown	106 000	1 620 000	15.30
Total	841 000	$15 490 000	$18.40

These works appear to have been very successful in affording the designed protection to the Miami Valley. Their effect on the Ohio and Mississippi floods is to decrease the flood flow for 1 day, or 2 days, and then increase it by a somewhat smaller amount for several days. Whether the net result is to increase or decrease the maximum flow of the larger river is largely a matter of chance.

This effect was computed for the years of the last five great Mississippi floods. Table 17 shows the results on the Ohio River.

TABLE 17.—Effect of Miami Reservoirs on Ohio River Floods.

Year.	Maximum Natural Discharge.		Maximum Discharge, with Reservoir Operating.		Effect of Miami reservoirs, in second-feet.
	Quantity, in second-feet.	Date.	Quantity, in second-feet.	Date.	
1912	558 000	March 30	553 000	March 30	5 000, decrease
1913	868 000	March 31	890 000	March 30	22 000, increase
1916	520 000	April 2	520 000	April 2	No effect
1922	540 000	March 19	540 000	March 19	No effect
1927	396 000	March 26	398 000	March 22	2 000, increase

It appears from Table 17, therefore, that these reservoirs have no dependable effect in reducing the magnitude of maximum floods of the Ohio River and that, in some instances, the effect may be adverse.

The effect on the floods of the Mississippi is dependent on the time required for the effects at the mouth of the Miami to be felt at Cairo, Ill. This has been taken to be nine days. If the effect of "valley storage" in the Ohio between the Miami and Cairo be neglected, the results are as given in Table 18.

TABLE 18.—EFFECT OF MIAMI RESERVOIRS ON MISSISSIPPI RIVER FLOODS.

Year.	Maximum Natural Discharge.		Maximum Natural Discharge, with Miami Reservoirs Operating.		Difference, in second-feet.
	Quantity, in second-feet.	Date.	Quantity, in second-feet.	Date.	
1912	1 946 000	April 5	1 945 000	April 5	1 000, decrease
1913	1 677 000	April 4	1 725 000	April 7	48 000, increase
1916	1 903 000	February 5	1 903 000	February 5	No effect
1922*	1 582 000	April 25	1 584 000	April 25	2 000, increase
1922*	1 595 000	March 25	1 596 000	March 25	1 000, increase
1927	1 800 000	April 19	1 800 000	April 19	No effect

* There were two flood peaks of nearly equal magnitude at Cairo in 1922.

Table 18 shows that the effect on the Mississippi may be either beneficial or the reverse. The effect of the "valley storage" on variations of flow at the mouth of the Miami of only a few days' duration is such that the actual results at Cairo would be similar to those in Table 18, but of less magnitude.

If these Miami reservoirs, instead of being used as detention basins for the protection of that valley, had been used as reservoirs for the protection of the Mississippi under the operating schedule adopted by the Reservoir Board, their effect would have been to reduce the crest at Cairo by quantities varying between 1 000 and 7 000 sec-ft. for different flood-years. The dependable effect would not exceed 0.2 in. on the Cairo gauge. If the reservoirs were used in this manner a new method of protection for the Miami Valley would have to be provided, and the total cost probably would be in excess of $30 000 000.

KEOKUK RESERVOIR

As now operated, the Keokuk Reservoir has no measurable effect on Mississippi floods. If operation were modified in the interests of flood protection the pond might be drawn down 6 ft. below the normal stage by February 5, and the resulting 142 800 acre-ft. filled uniformly over the probable 90-day flood period. This would give a reduction of 790 sec-ft. on Mississippi floods, and would lower the flood height at Cairo by less than $\frac{1}{7}$ in. It would entail a power loss of 16 000 000 kw-hr., the value of which, if replaced by steam power, is greater than the cost of obtaining similar flood benefit by other means.

The level of the pool cannot be raised more than 3 ft. without excessive flowage damages. It might be raised 3 ft., giving 100 000 acre-ft. additional storage, at an estimated cost of about $5 200 000, or $52.00 per acre-ft. Such a

storage would reduce the flood flow about 560 sec-ft. and the crest height at Cairo about 0.1 in. The cost would be more than ten times as much as the cost of levees to take care of the same flow at the top of the flood.

Reclamation Service Reservoirs

The U. S. Reclamation Service has built a number of large irrigation reservoirs in the Mississippi Drainage Basin. To determine what effect they have had on Mississippi floods, data are available for eight of these reservoirs for 1922 and 1927, as shown in Table 19.

TABLE 19.—Irrigation Reservoirs in Mississippi Drainage Basin.

Name.	Capacity, in acre-feet.	Approximate cost.	Cost, per acre-foot.
Pathfinder	1 020 000	$2 000 000	$ 2
Lake Mintare	60 800	500 000	8
Lake Alice	12 000	100 000	8
Winter Creek	3 700	100 000	27
Belle Fourche	203 000	1 200 000	6
Willow Creek	17 000	200 000	12
Shoshone	457 000	1 400 000	3
Nelson	40 000	200 000	5
Total	1 813 500	$5 700 000	$3.15 (average)

At the time when the water that was to contribute to the crest of the 1927 flood was passing these sites the Shoshone Reservoir was discharging 550 sec-ft. more than normally. The Pathfinder and Belle Fourche Dams were storing at moderate rates, about 600 and 250 sec-ft., respectively. At the other sites, stream flow and storage were both very low. It should be noted that the time in question comes in March, before the first spring thaws occur in this mountainous northern country. The aggregate storage rate for the eight reservoirs is estimated at 325 sec-ft.

In 1922 the flood had two practically equal crests at Cairo; one late in March and one late in April. From the earlier one the irrigation reservoirs held back about 900 sec-ft., while from the later they withheld more than 3 200 sec-ft.

The quantity held back in the 1927 flood was equivalent to about $\frac{1}{16}$ in. on the gauge at Cairo. This is based on the actual operation of the irrigation reservoirs. With the exception of Shoshone, it appears that they were storing practically all the water that there was and that the volume could not have been materially increased. The Shoshone Reservoir was being used for the joint purpose of irrigation and power development. Had it been operated primarily for Mississippi flood control, it could probably have stored about 500 sec-ft.

Reservoirs at Head-Waters of the Mississippi

In the State of Minnesota, on the head-waters of the Mississippi River six reservoirs are maintained by the United States. They are operated pri-

marily for the purpose of increasing the navigable depths in the Upper Mississippi above Lake Pepin during periods of low water. They also serve to increase the amount of water power developed during these periods and to diminish the damage caused by local floods. These purposes are accomplished in a fairly satisfactory manner; but ever since the reservoirs were finished the War Department has been receiving frequent demands to operate them in some other way to benefit other interests.

The cost of these reservoirs was very small as they were formed by the construction of low dams at the outlets of natural lakes at a time when price levels were down and land and flowage rights were cheap, as the country was largely undeveloped. The District Engineer at St. Paul reports the capacities and costs as in Table 20.

TABLE 20.—HEAD-WATER RESERVOIRS, MISSISSIPPI RIVER.

Name.	Capacity, in acre-feet.	Cost.	Cost per acre-foot.
Lake Winnibigoshish	967 170	$368 200	$0.38
Leech Lake	743 340	246 800	0.33
Lake Pokegama	120 760	197 000	1.63
Sandy Lake	72 500	218 000	3.00
Pine River Reservoir	177 520	215 500	1.21
Gull Lake	70 970	77 200	1.08
Total	2 152 226	$1 322 700	$0.61

The effect of the operation of these reservoirs in reducing the floods of the Lower Mississippi was studied for each of the great floods of recent years with the following results.

Flood of 1912.—In this year the flood crest reached Cairo in the early part of April and the operation of the Minnesota Reservoir System caused a reduction of about 1 100 sec-ft., or less than 0.2 in. on the gauge at Cairo.

Flood of 1913.—The flood crest reached Cairo early in April. The Upper Mississippi was at a low stage at the time corresponding to this flood wave, and the reservoirs were discharging water in order to maintain navigable depths. This resulted in increasing (rather than decreasing) the flood discharge by about 230 sec-ft.

Flood of 1916.—The flood crest reached Cairo early in February. Conditions in the Upper Mississippi were similar to those in 1913 and the operation of the reservoirs increased the flood discharge about 250 sec-ft.

Flood of 1922.—This flood had two approximately equal crests at Cairo, the first late in March and the second late in April. The operation of the reservoirs reduced the earlier crest by 500 sec-ft., and the later crest by 1 700 sec-ft., or about ⅓ in. on the gauge at Cairo.

Flood of 1927.—The flood crest reached Cairo just after the middle of April. Operation of the reservoirs caused a reduction of about 1 200 sec-ft., or slightly more than 0.2 in. on the gauge at Cairo.

In 1912, 1922, and 1927, the reservoirs were storing the entire flow of their streams except the minimum that had to be passed to prevent damage to

navigation and power development. If they had been operated solely for the benefit of the Mississippi floods they could have done no more without damage to these interests. In 1913 and 1916, they were contributing a small increase to the Mississippi floods to the benefit of navigation and power.

Summary.—Leaving out the Miami regulation which is more apt to be detrimental than beneficial the three propects just discussed have a combined storage capacity of about 4 150 000 acre-ft. As now operated their combined dependable reduction of Mississippi flood peaks is about 75 sec-ft.; that is, to reduce past floods 1 sec.-ft. has required more than 55 300 acre-ft. of storage. If these reservoirs had been operated primarily for Mississippi flood protection they might have reduced Mississippi floods by 2 200 sec-ft., or nearly thirty times as much. These ratios are interesting, but should not be used indiscriminately.

Possible Reservoirs on the Tributaries of the Mississippi River

For this next study the water-shed of the Mississippi was divided into five principal divisions, as follows: (1) The Ohio River and its tributaries; (2) the Upper Mississippi River; (3) the Missouri River; (4) the Arkansas River group of tributaries; and (5) the Red River group of tributaries.

The Upper Mississippi has been taken to include that part of the river between St. Louis, Mo., and Cairo. The Arkansas group includes the White, Black, and St. Francis Rivers and minor tributaries between Cairo and Arkansas City, Ark. The Red River group includes the Ouachita and Yazoo Rivers and neighboring small streams. Each of these divisions differs from the others in its run-off characteristics and also in its effect on Mississippi floods. Table 21 shows the drainage area of each division and the percentage of flood flow contributed by each in recent floods.

TABLE 21.—Principal Tributaries of Mississippi River with Drainage Areas and Percentage of Flows Contributed.

Division.	Area. in square miles.	Percentage of Flow at Cairo, Ill.					
		1912.	1913.	1916.	1922*.	1927.	Average.
Ohio	203 000	61	72	57	70 57	46	60
Upper Mississippi	186 000	23	25	32	19 28	36	28
Missouri	530 000	16	3	11	11 15	18	12
		Percentage of Flow at the Mouth of the Arkansas River.					
Arkansas	202 000	12	9	17	19	40	19
		Percentage of Flow at the Mouth of the Red River.					
Red	104 000	13	8	13	15	14	13

* In 1922 there were two nearly equal flood crests at Cairo.

The percentages of flow at Cairo are fairly accurate, but those at the Arkansas and Red Rivers are rather uncertain, due to difficulties in determining the actual flow in the Mississippi at these points.

The District Officers reported on all reservoirs that could be found with a capacity of 100 000 acre-ft. or more. In all, 321 reservoirs were included in the reports. Of these 124 were thrown out as impracticable, or unnecessary, chiefly because there was not sufficient water to fill them at the time they are needed.

Table 22 gives a summary of the 197 reservoirs which have been studied in detail.

TABLE 22.—SUMMARY OF POSSIBLE FLOOD CONTROL RESERVOIRS.

Division.	Number of reservoirs.	Total capacity, in acre-feet.	Total cost.	Average cost per acre-foot.	Percentage of drainage area above lowest reservoir.	Percentage of total rainfall of Mississippi River above lowest reservoir.
Ohio	90	29 200 000	$543 000 000	$18.60	30	Not available.
Upper Mississippi	54	17 400 000	305 000 000	17.50	86	
Missouri	7	32 700 000	232 000 000	7.10	84	
Arkansas	32	23 500 000	201 000 000	8.60	58	
Red	14	8 700 000	32 000 000	3.70	45	
Total	197	111 500 000	$1 313 000 000	$11.80	65	45 in 1927

From the hydrographs and storage curves of the reservoirs, graphs were prepared showing the aggregate storage rate in the reservoirs of each of the five divisions for each day of the 90-day storage period. The actual reduction of flow in the Mississippi Valley is quite different from the sum of the rates of storage at the various reservoirs, because of the retarding and equalizing effect of the storage in the valleys of the tributaries between the reservoirs and the Mississippi. For example, between high and low water the Ohio has a storage capacity in the main valley of about 12 000 000 acre-ft., more than 40% of the capacity of all the reservoirs available on its water-shed.

Computations to determine the effect of this valley storage are not yet completed. A rough approximation, therefore, has been used to obtain the figures in the computations to follow. The reduction in maximum flood discharge that would have been brought about by the reservoirs of each division is shown approximately in Table 23. In Table 23, as in the previous tables, the results at Cairo are fair, but those at points below are not so good, owing to doubt as to the flow in the Mississippi River at these lower points.

The effect on gauge heights of this reduction of discharge depends on the increment of discharge corresponding to a change of stage of 1 ft. near the crest of high confined floods. The increment of discharge varies considerably

in different floods due principally to variations in the slope of the river caused by variations in relative contributions by the tributaries. Studies on this subject are still in progress, but it is believed that the following discharges can be taken as fairly safe: (*a*) 65 000 sec-ft. at Cairo; (*b*) 80 000 sec-ft. at the mouth of the Arkansas River; and (*c*) 75 000 sec-ft. at the mouth of the Red River. The last quantity is a joint increment for the Lower Mississippi and Atchafalaya Rivers.

TABLE 23.—NET MAXIMUM FLOOD DISCHARGE AS REDUCED BY RESERVOIRS, IN SECOND-FEET.

Division.	1912.	1913.	1922.	1927.	Minimum.
Ohio	195 000	210 000	200 000	140 000	140 000
Upper Mississippi	80 000	85 000	80 000	95 000	80 000
Missouri	140 000	90 000	90 000	110 000	90 000
Total at Cairo	415 000	385 000	370 000	345 000	345 000
Arkansas Group	120 000	45 000	130 000	250 000	45 000
Total at mouth of Arkansas	600 000	340 000	500 000	600 000	340 000
Red River Group	20 000	8 000	35 000	50 000	8 000
Total at mouth of Red River	430 000	250 000	420 000	450 000	250 000

Table 24 shows the approximate dependable effect reservoirs would have had in reducing maximum flood heights that have heretofore occurred.

TABLE 24.—EFFECT OF RESERVOIRS IN REDUCING MAXIMUM FLOOD HEIGHTS.*

Division.	At Cairo, in feet.	At mouth of Arkansas River, in feet.	At mouth of Red River, in feet.
Ohio	2.15	1.75	1.88
Upper Mississippi	1.23	1.00	1.08
Missouri	1.38	1.12	1.21
Total at Cairo	5.30		
Arkansas Group	....	0.56	0.61
Total at mouth of Arkansas River	....	4.25	....
Red River Group	....	....	0.11
Total at mouth of Red River	....	....	8.36

* The total effect of several divisions on the same guage is not the same as the sum of their effects on that gauge.

The estimated cost of reservoirs to produce this lowering of gauge heights is $1 313 000 000.

In Table 25 is shown the cost of reducing the Mississippi River discharge 1 sec-ft. by reservoirs in each of the five divisions.

The studies of other boards working on the Mississippi flood protection problem indicate that it is feasible to protect the valley against the maximum probable flood by levees, spillways, and by-passes, which means that reservoirs are not essential to a Mississippi flood project and, consequently, their adoption or rejection should rest on their economic value. The studies have not reached a point where final results can be given; but an estimate has been made of the difference in cost of protecting the valley by levees against the

maximum probable flood and against a flood of 300 000 sec-ft. less. The results are probably higher than the cost of any alternate method that might be adopted, such as the provision of a by-pass through Tensas Basin, or of a large amount of storage in St. Francis Basin. The results are given in Table 26.

TABLE 25.—Cost of Reducing Mississippi River Discharge 1 Second-Foot by Reservoirs.*

Division.	Reservoir capacity, in acre-feet.	Cost of reservoirs.	Dependable reduction, in second-feet.	Cost per second-foot of reduction.
Ohio..........	29 200 000	$543 000 000	140 000	$3 870
Upper Mississippi......	17 400 000	305 000 000	80 000	3 800
Missouri.........	32 700 000	232 000 000	90 000	2 590
Arkansas Group......	23 500 000	201 000 000	45 000	4 490
Red River Group......	8 700 000	32 000 000	8 000	4 020

* Reservoirs in the Arkansas and Red River groups reduce the flow only in that part of the Mississippi River lying below them, respectively.

From Table 26 it appears that a reduction of flow at Cairo by 1 sec-ft., is worth $860. The cost of obtaining it by reservoirs on the Ohio would be $3 870, or 4.5 times as much; on the Upper Mississippi, $3 800, or 4.4 times as much; and on the Missouri, $2 590, or 3 times as much.

TABLE 26.—Saving in Cost of Flood Protection by Levees Due to a Reduction of 300 000 Second-Feet in Maximum Flood Flow.

Part of river.	Estimated cost.	Cost per second-foot of reduction.
Cairo to mouth of Arkansas..................	$ 27 000 000	$ 90.00
Mouth of Arkansas River to mouth of Red River..	192 000 000	640.00
Mouth of Red River to Gulf..................	39 000 000	130.00
Total	$258 000 000	$860.00

A reduction in the discharge of the Arkansas group of 1 sec-ft., is worth $640 plus $130, or $770. The cost of obtaining this by reservoirs is $4 490, or 5.8 times as much. A similar reduction on the Red River group is worth only $130 and the cost of obtaining it by reservoirs is $4 020, or 31 times as much.

This relation can also be set up by comparing the cost of an acre-foot of storage with the cost of an equivalent amount of protection by levees. Table 27 shows such a comparison.

Columns (5) and (6), Table 27, show the costs, which should be compared with the values in Column (4). It appears that only in the Upper Mississippi Division are any reservoirs found the cost of which is less than their computed values for flood protection. Four such reservoirs are found in that Division. Their combined capacities total about 846 000 acre-ft.; and they would lower the gauge height at Cairo about ¾ in. A further analysis of them is being

made to see what would be the effect on other interests of taking these reservoirs for Mississippi flood protection.

TABLE 27.—COMPARISON OF COST OF AN ACRE-FOOT OF STORAGE TO COST OF EQUIVALENT LEVEE PROTECTION.

Division. (1)	Value of 1 sec-ft. (2)	Acre-feet required for 1 sec-ft. (3)	Value of 1 acre-ft. (4)	Average cost of 1 acre-ft. (5)	Cost of 1 acre-ft. in cheapest reservoir. (6)
Ohio	$860	208	$4.13	$18.60	$6.00
Upper Mississippi	860	217	3.96	17.50	2.50
Missouri	860	364	2.36	7.10	4.22
Arkansas Group	770	522	1.48	8.60	3.94
Red Group	130	1 087	0.12	3.70	1.97

The studies of the Board have showed that many reservoir sites have an economic value for local use that is greater than their value for Mississippi flood protection, so that before any reservoirs are appropriated exclusively for Mississippi flood protection, a careful analysis should be made to ascertain what the effect of such action would be on local interests. On the assumption that the best interests of the country will be served only when reservoirs are used to produce their maximum economic value, it may be concluded that if reservoirs have a large potential local value they should not be lightly appropriated for exclusive Mississippi flood use. It has been found that operation of reservoirs for local purposes will generally greatly reduce their value for Mississippi flood protection, and, in fact, may even make that value a negative quantity, as already shown in the case of the Miami Conservancy Reservoirs and the reservoirs at the head of the Mississippi.

The costs of reservoirs noted in this paper are the total costs as estimated by the District Engineers. In some cases benefits would result to other interests, which might somewhat reduce the costs to be charged to Mississippi flood protection. The estimates of such benefits are not yet complete, but the indications are that, while they may justify further analysis of a few reservoirs, they will not materially change the general results.

RESERVOIRS FOR COMBINED USE

In addition to the studies of reservoirs to be built and operated primarily for Mississippi flood protection, the Board has made extensive studies of the results to be obtained should reservoirs be built and operated for their greatest economic value, considering all uses. Obviously, the time and funds available were not sufficient for completing such studies for all the streams in the Mississippi Drainage Basin. There are almost an infinite number of ways in which any stream can be developed and operated. The Board, therefore, concentrated its efforts on the streams on which other interests had proposed and studied developments. The results of its studies are not yet in such shape that a complete digest of them can be presented; but some of the results can be indicated.

In general, it appears that the values of reservoirs for local flood protection, irrigation, power, or navigation, or for a combination of such uses, is much greater than their value for Mississippi flood protection. In other words, Mississippi flood protection, if proper weight be given to economics, will generally be a minority interest in reservoir development, and the practical question arises as to how far a minority interest can go toward securing participation of the majority interests. Such an undertaking can be accomplished in only two ways; either the minority interest must wait till the majority interest is ready to go, or it must undertake the financing for both interests and assume the risk of being able to profitably dispose of the majority interest after development is made.

There may be some streams on which the majority interests will be ready to go before the Mississippi flood project is carried out, and on such streams it may be practicable to secure a certain amount of Mississippi flood protection by contribution commensurate with its value. On the other hand, the need for Mississippi flood protection is immediate, and its cost, at best, promises to burden the Treasury with all or more than it should properly bear. There will doubtless be advocates ready, as in the past, to use the urge for Mississippi flood protection as a means to induce the Federal Government to undertake comprehensive projects for complete development of some or all of the streams, regardless of what proportion of the benefits may accrue to Mississippi flood protection. Such projects should be studied and treated on their own merits, with due consideration to all interests; and it is to be hoped they will not be permitted to delay action on a Mississippi flood project should it not include them.

Thus far, the Board has found no reservoir projects that it feels justified in recommending for combined development, but, as stated previously, it recognizes that such projects may develop before the Mississippi flood project is too far completed to make use of them. At such time, it will not be difficult to reach a conclusion as to what use can be made of them.

Otherwise than those mentioned, it is believed that dependable benefits to Mississippi floods that may result from the development of reservoirs for combined uses, will not be large and that they should be counted on solely as contributing to the factor of safety on whatever Mississippi flood project is adopted. The Board is endeavoring to arrive at a measure of the probable benefits to Mississippi flood protection should the maximum economic development of all streams in the drainage basin be accomplished. At best, such an estimate cannot be accurate and perhaps the nearest approach to an answer will come from determining the effect on Mississippi floods of equating the flow of the tributaries as nearly as the available reservoir capacities will permit.

From the studies to date, it appears that it may not exceed 10% of the effect of the same reservoirs used primarily for Mississippi flood protection.

Valley Reservoirs

In addition to reservoirs on tributaries, the Board was charged with the duty of determining whether any feasible reservoirs could be found in the Valleys of the Mississippi and Ohio themselves.

The results of the studies on this subject are not yet completely available, but it appears that reservoirs could be built on the main river at Commerce, Mo., in the St. Francis Basin, at the mouth of the Yazoo, and in a few other places. No practicable reservoir sites were found in the main Valley of the Ohio.

The cost of the reservoir at Commerce, Mo., appears to be too great to make it feasible at this time, even when credited with all possible benefits from power navigation. The reservoirs at the mouths of the lower tributaries, such as the Yazoo, do not offer much promise. The St. Francis Basin offers the best reservoir possibilities in the valley. To create a reservoir there, however, will drown out several towns and necessitate the relocation of many miles of railroad and highway. The estimates of the flowage damages are not yet complete, and until available, no opinion as to the feasibility of the project is justified.

To conclude, the prospects for reservoirs as a part of a Mississippi flood project, are not very good. One after another of the possibilities have "washed out" till practically nothing is left, except certain relatively small reservoirs on the Upper Mississippi and certain reservoirs in the Mississippi Valley proper or at the mouths of the principal tributaries and even they do not look very promising.

The Board, urged on by the Chief of Engineers, has used every effort to find feasible reservoirs. Its studies have been far-reaching and while the limited time available has made it necessary to adopt certain "short cuts" and approximations, it is the writer's belief that the results will be sufficiently accurate to permit a dependable determination of the extent to which reservoirs may enter into the Mississippi flood project to be adopted.

THE BASIS OF THE CASE AGAINST RESERVOIRS FOR MISSISSIPPI FLOOD CONTROL

By Arthur E. Morgan,* M. Am. Soc. C. E.

There has been a fifty-year controversy between the proponents of reservoir control and of levee control for Mississippi floods. The writer is not attempting to settle that issue. He does not know what are the aggregate possibilities for reservoir control, and any estimate made without very extensive and thorough-going investigation can be but a guess.

It is a fundamental axiom of sound engineering that when great issues are at stake, no conclusion of large importance should be left to chance. Actual and definite studies of actual cases are necessary. Until they have been carried to a point where, as nearly as is reasonably possible, all major possibilities have been conclusively determined, there is no adequate basis for far-reaching engineering policy.

If, as is contended by the members of the Mississippi River Commission, the case against reservoir control for the Mississippi has been properly established, it is because a definite study of the matter has been made, and because a conclusion has been reached in accordance with the evidence. The transmission of an authoritative dogma from one generation to another will not suffice. Somewhere along the way that authoritative doctrine must have been established by scientific methods, and those methods and results must be subject to re-examination.

The writer decided that the best contribution he could make to this discussion was to try to discover the exact nature of the evidence which has led the Mississippi River Commission to reject reservoirs. He has read a vast amount of literature on the subject, especially the writings of members of the Commission and of Army Engineers, and he has constantly watched for references to other and authoritative opinions and reports.

The whole controversy has been clouded by two factors. First, it has been made to appear as an issue of levees *versus* reservoirs, on the assumption that they are mutually exclusive.

Such is not the case. In a great problem like that of the control of the Mississippi it would be almost miraculous if 100% of the desired results could be achieved by only one method. The real problem is to discover and appraise all possibilities, and then, by careful analysis and synthesis, to design a total improvement in which each method is introduced to whatever degree is most effective and economical. If even 10% of the desirable results can be secured by each of two or three other methods, then control by levees may be reduced to far more manageable limits.

So far as the writer can discover, up to 1927 this attitude of endeavoring to work out a solution that will use all desirable methods never has been taken by the Mississippi River Commission, but rather the attitude of an armed camp in the battle of levees and revetment against all other methods of control.

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Second, great confusion has been brought into the controversy by the absence of a basis for comparison. There frequently has been a general admission that reservoir control would be preferable to levee control if it were economically as feasible. To determine that levee control is less expensive it is necessary to know approximately what each would cost. Now, neither of these costs ever has been adequately determined. The Mississippi River Commission estimates of the discharges to be provided for by a levee system, and of the works ultimately necessary, always have been made by arbitrary "practical" methods, which by all the laws of probability were bound to fail. The writer agrees with the statement of N. C. Grover, M. Am. Soc. C. E., that students of hydrology have long known that a flood of the magnitude of that of 1927 was inevitable.* The failure to recognize flood possibilities is one of the causes of inadequate estimates.

That the official estimates have been presented to the public as the full cost of levees in a program of complete protection, there can be no doubt. The report of the Special Engineering Board of 1874-75, which led to the establishment of the Mississippi River Commission, largely established the policies by which it has been controlled. This report indicated a cost of less than \$46 000 000 for complete control by levees.

The late H. M. Chittenden, M. Am. Soc. C. E., at one time Chief of Engineers U. S. Army, formerly was a recognized authority of the Army Engineers against reservoirs. In deciding against reservoir control on the basis of cost, he referred to the report of the Mississippi River Commission for 1896, as authority for the statement: "Take \$40 000 000 and reinforce the entire levee system of the Mississippi. That will make it impregnable. * * *"†

In 1896 the levee system was only well begun. If \$40 000 000 would make it impregnable, then it was almost certain that reservoir control would be more expensive.

In the 1896 report of the Chief of Engineers, U. S. Army, the estimated cost of complete protection by levees is \$18 000 000. In 1912, the President of the Commission stated that \$73 000 000 was ample to do all remaining necessary work.‡ In 1922, a member of the Commission stated:§

> "There is no uncertainty as to the outlay that will be necessary to * * * complete flood control * * * neither is there material uncertainty as to the grade line to which the levees must be constructed * * * no increase in height is now contemplated, except to meet local conditions such as are due to shifting of the channel through chutes or cut-offs."

In 1924 the Chief of Engineers stated‖ that 90 000 000 cu. yd. would complete the work. In 1925, in an article entitled, "Why the Mississippi River Commission Sticks to the Policy of Levees Only",¶ the President of the Commission stated:

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* See p. 2489.

† *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 304.

‡ *Engineering Record*, October 26, 1912, Vol. 66, p. 471.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1479.

‖ *Loc. cit.*, Vol. LXXXVII (1924), p. 976.

¶ *Engineering News-Record*, Vol. 94, p. 513.

"There are now completed or nearing completion 1824 miles of levee line on the Mississippi River, containing 429 000 000 yards of earth, and requiring about 76 000 000 yards to finish it. That would have been the end of the Commission's levee building but for the fact that Congress has recently extended the Commission's jurisdiction to the tributaries and outlets. * * *"

Year by year the Commission has reported the approach of the levee system to final completion. Under the heading "Effect of Improvement", the Commission formerly reported:*

"It may be stated that in a general way the improvement [of the Mississippi] is providing a safe and adequate channel for navigation and is now in condition to prevent the destructive effects of floods in all except the most extreme high waters. * * *"

For the past three years, beginning with the report of 1924, this qualifying clause has been omitted, and the statement reads: "* * * is now in condition to prevent the destructive effects of floods".†

The writer believes he has quoted enough to indicate that the estimates of the Mississippi River Commission have been inadequate. These estimates of the cost necessary to complete the levee system seldom have been more than 20% of the true cost, and sometimes less than 10 per cent. The writer believes that this extreme inadequacy of its estimates has rendered difficult, if not impossible, any dependable comparison between the cost of levee protection and of reservoir protection.

On the other hand until recently there has not been even a remote effort on the part of the Commission to discover and to estimate the cost of the most favorable opportunities for reservoir control. Thus, we have the remarkable situation of fifty years of insistence of the superiority of levees over reservoirs, without a thorough-going engineering analysis of the cost of either.

Perhaps, however, the members of the Mississippi River Commission are so thoroughly informed concerning the principles and facts of reservoir control that their informal judgment is sufficient. The writer has searched their writings of nearly half a century to find evidence of that clear understanding, but he finds it to be almost completely lacking. They have many times given their reasons for rejecting reservoirs, the more important of which will be mentioned.

In 1912, the President of the Mississippi River Commission stated‡: "To control the flow of every stream in the Mississippi Valley by reservoirs is a pretty large job, even for the United States Government, but that is what the control of the Mississippi during floods by reservoirs signifies." This statement, of course, is incorrect. A similar impression is repeatedly given by referring to the vast drainage area of 1 250 000 sq. miles to be controlled. In fact, although the Ohio drains but one-third of the drainage area of the Mississippi above Cairo, Ill., and but 16% of the drainage area of the whole river, it delivers from 55 to 85% of the flood waters that pass Cairo. The control of that third of the water-shed above Cairo would go far toward furnishing all the control necessary at Cairo, and for a considerable distance

* Rept., Chf. of Engrs., U. S. Army, 1923, Vol. I, p. 1862.

† *Loc. cit.*, 1926, Vol. I, p. 1793.

‡ *Engineering Record*, Vol. 67, May 3, 1913, p. 506.

below. These men reiterate that the Ohio supplies only a third of the total annual run-off at Cairo, as if that statement were significant. The fact that it delivers 55 to 85% of the flow of great floods at Cairo is the important item.

In 1922, another member of the Mississippi River Commission stated* that "this shifting of the locus of great floods over such great areas prevents reservoir control." In illustration he mentions five floods originating in the Ohio and mentions the floods of 1844 and 1903 as originating in the Mississippi above Cairo. This statement is misleading, for both the floods he mentioned as coming from the Mississippi above Cairo, although they were the two record floods on the upper river, before 1927, did not furnish enough water to be of importance below the mouth of the Ohio. The 1903 flood, which did such damage below Cairo, was not the summer flood from the Upper Mississippi, but a spring flood from the Ohio.

We are repeatedly informed that the Commission and the Army Engineers know the value and limitations of reservoirs because they built and operate those on the Upper Mississippi in Minnesota.† These Minnesota reservoirs referred to by both friends and enemies of reservoirs as "the greatest system of reservoirs in the world", have practically no significance in the problem of flood control for the Mississippi, except to a minor degree within the State of Minnesota. They control an exceedingly small part of the total flow, and are too far removed from the lower river to have any appreciable effect on floods. What the Mississippi River Commission has learned about flood control by the operation of the Minnesota reservoirs is nothing of importance to the lower river.

This same President of the Commission makes a statement that is frequently repeated by the Army Engineers, and has been taken by them almost as an axiom. It is that because of the lack of sites lower down, flood-control reservoirs must be limited to the head-waters of streams.‡ Flood-control reservoirs are best located low down on the streams, as nearly as possible just above the property they are to protect. Contrary to the frequently repeated statements of the members of the Mississippi River Commission, suitable sites low down on the main tributaries probably do exist.

Many engineers look back to the classic paper against reservoirs by Gros, Inspector-General of the Department of Bridges and Roads of France, in which the same assumption was made that dams must be limited to head-waters. This being almost the only European paper on the subject published in the American technical press until recent years, it is almost the only one referred to in their discussion.§

This paper, first published in 1881, was then reasonably sound in view of the existing limitations of dam design, although it is partisan. In those days French dams must be of cut-stone masonry, founded on solid rock. To-day

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1462.

† *Engineering Record*, Vol. 66, p. 470; *Engineering News-Record*, Vol. 94, p. 557.

‡ *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 351; *Engineering Record*, Vol. 66, p. 470.

§ *Engineering News*, Vol. 25, March 14, 1891, p. 258.

some of Gros' most serious objections are of small importance. The dams of the Miami Conservancy District would have been totally beyond his horizon. Engineering design has been revolutionized, and the valleys of large rivers often furnish safer and better sites for great dams than the head-waters.

A far more important French discussion of about the same period is that by Lechalas in "River Hydraulics", published in Paris in 1884,* although it, too, is partly obsolete.

The assumption that sites for flood-control dams can be found only on head-waters permeates the thinking and writing of the Army Engineers and members of the Mississippi River Commission, and largely destroys any value their studies might otherwise have had.

A President of the Mississippi River Commission, who served as Chairman of the Special Committee on Floods and Flood Prevention of the Society, states that all the reservoirs of a certain suggested system on the Upper Ohio would hold the total flood flow of the Mississippi for only one day, and that, as a flood may last 40 days, it is implied that forty times as much reservoir capacity would be necessary.† This kind of reasoning is found throughout the discussions of members of the Mississippi River Commission. It represents one of the defects in the current reasoning on the subject. Even in the great flood of 1927, a holding back of 20% of the total flood flow probably would have been more than adequate. While the excess flow at the crest of the flood may be one-half or more of the total flow, the crest flow lasts for only a small portion of the total flood period. It should be recognized, however, that reservoir storage never can be perfectly timed to hold back the crest only, and that therefore reservoirs seldom operate with the highest theoretical efficiency.

Members of the Commission and Army Engineers repeatedly state that reservoirs on the Ohio System would soon fill with silt and be useless,‡ yet a careful study by Stabler, with reference to a proposed system of reservoirs on the Ohio, indicated that if all the silt above them were held, they would lose less than 10% of their capacity in 700 years.§ In flood-control retarding basins like those of the Miami Conservancy District, not more than one-third of the annual silt load is held in the basins, so that the life of a retarding basin on the Ohio would run into thousands of years before 10% of its capacity would be lost.

Another member of the Mississippi River Commission states,|| "Nature has greatly simplified the problem of flood control by making the drainage of the flooded lands, in the larger basins, comparatively simple." It is a fact that there are few more difficult problems of drainage and flood control than are found in these same basins. In the great St. Francis and Yazoo Basins there are six or more streams emerging from the hills, each of which has a maximum flood flow of 100 000 to 150 000 cu. ft. per sec., with an initial

* "Théorie Général des Réservoirs d'Emmagasinement des Crues" from "Hydraulique Fluviale," by Lechalas, p. 434, Baudry et Cie., Paris, 1884. The Dayton-Morgan Engineering Co. has a translation of this article, prepared by K. C. Grant, M. Am. Soc. C. E.

† *Engineering Record,* Vol. 66, p. 471.

‡ *Engineering News,* Vol. 60, p. 378; *Transactions,* Am. Soc. C. E., Vol. LXII (1909), p. 351.

§ *Loc. cit.,* p. 649, 1908.

|| *Transactions,* Am. Soc. C. E., Vol. LXXXV (1922), p. 1467.

channel capacity in the basin of less than 3% that amount. Control of these streams by reservoirs is feasible and probably is the only feasible method within reasonable cost. Such reservoirs would help solve the general Mississippi problem, and without them much of the land protected by levees will still be subject to destructive local flooding after the great river is controlled. In the writings of the Army Engineers and of the Mississippi River Commission for forty years one finds scarcely a hint of this great problem. It nearly always has been ignored in their general plans.

Repeatedly, we have the objection that reservoirs must not be built because of the valuable farm land that would be flooded. In all the discussion of the subject by members of the Commission one sees no inclination to offset this loss by the gain of a greater area of fertile land along the Mississippi between the river channel and the levees. In referring to the valuable storage along the river, a President of the Mississippi River Commission states that the levees are "several miles apart", the river channel being somewhat less than a mile wide; but while he denounces the waste of land by reservoirs, he does not mention this greater loss of fertile land along the river. Much of the reservoir area would be river bed or rough wooded hillside, whereas most of the land along the Lower Mississippi between the river channel and the levees is very fertile, sandy loam. In this item, which is repeatedly referred to as a controlling objection, the gain by reservoirs would wholly or largely offset the loss in the reservoir areas.

The Miami Conservancy Basins have been in operation eight seasons. All the bottom-land above the dams continues to be farmed, and only a small fraction of 1% of the crops planted in them has been lost by the operation of the dams. The lands increase in fertility through the deposit of silt in early spring floods, and steadily increase in productiveness. The sweeping generalizations of members of the Commission on this point are unsound.

Repeatedly the members of the Commission and Army Engineers condemn reservoirs because of the danger of failure of dams. This objection does not run against flood control alone, but against water supply, irrigation, and power development as well. Well-built dams are a permanent part of our civilization.

The disorganization of highways, railroads, and towns is repeatedly given as a conclusive objection to flood-control dams. Until definite studies are made these indefinite misgivings have little value in determining policies.

For the plans of the Miami Conservancy District, storage space was needed on the Mad River. The best dam site available was across a flat valley nearly a mile wide, with gravel and sand a hundred feet deep below the surface. Two important trunk-line railroads crossed the dam site and traversed the length of the valley almost at water level, with a ruling grade of only 0.2 per cent. An electric line paralleled the two railroads, although with more tolerance as to grades, and an old established town of 1 200 people was in the proposed basin. Main-line and branch highways crossed the site from end to end. Yet it paid to overcome all these difficulties and to build a dam on that site in order to supply only one of the seven major elements of a local improvement. When we come to face the Mississippi River problem in its entirety it may be

found desirable to retard flood flows by the construction of dams under even greater difficulties and on a far larger scale.

Unsanitary conditions above dams are continually emphasized. The Miami Conservancy dams have not caused such conditions, and the storage areas in the Adirondacks are among the chief playgrounds of the nation. Floods have far more unsanitary effects in the low areas of Ohio River towns, and these conditions might be relieved by reservoirs.

It has been contended by Army Engineers that reservoirs would have to be paid for by the National Government, whereas flood control must be paid for locally. Their own policy on this point now seems to have changed.*

The dams of the Miami Conservancy District are repeatedly referred to as indicating the cost of reservoir flood control for the Mississippi. To secure the local benefits desired these dams could be built only above Dayton and Hamilton, Ohio, and mostly in a region thickly settled and traversed by many railroads and highways. More than a hundred public utilities were interfered with on that project. The President of the Mississippi River Commission† assumes that the whole cost of the Miami System was for dams and retarding basins, whereas a very considerable part was for channel control and levees. The high cost of channel improvement through the cities on that project could be used as an argument against levees and revetment as well as the cost of storing water can be used against reservoirs. For flood control on the Mississippi a much greater range of choice would exist. Even on the Miami River, a similar amount of storage for flood control on the Mississippi probably could have been secured at a far lower cost per acre-foot, and there would be a large amount of storage available. The writer is of the opinion that vast storage capacity is available well down on the Ohio and its main tributaries at a much lower cost than that of the Miami Conservancy District. Storage in the Miami Conservancy project cost about $18.00 per acre-ft., as compared with $2.65 per acre-ft., as an average for U. S. Reclamation Service projects.

The Miami dams were built for local flood control, and serve that purpose primarily. If they had been in operation during the 1913 flood they would scarcely have affected the flood stage exactly at the mouth of the Miami; but, because of the nature of the flood crest, due to increments lower down the Ohio, they would have reduced the crest stage at and below Louisville perhaps a foot. If the same dams with the same storage had been designed for Ohio River flood control they would have lowered the flood stage on the Ohio 3 to 5 ft. in a flood like that of 1913. The Miami Conservancy dams are more massive and more safely built than is customary, and would be adequate for any type of operation, so far as their strength is concerned.

The writer came to have an intimate acquaintance with the late H. M. Chittenden, M. Am. Soc. C. E., at one time Chief of Engineers U. S. Army. The writer feels free to refer to his writings because he has known but few engineers for whose intelligence, intellectual integrity, and persistent thoroughness, he has had such high regard, and because General Chittenden was recognized as an authority on this subject.

* *Engineering News,* Vol. 60, p. 378, October 8, 1908.

† *Engineering News-Record,* Vol. 94, p. 557.

In a paper delivered before the Society in 1908* he gave his reasons for disbelieving in the value of reservoirs for flood control. He refers to a system of "automatic reservoirs" proposed by the French Government on the Rhone about 1856. Evidently, he received his information from the previously mentioned paper by Gros, a translation of which had been published.† These reservoir dams were to have open outlets, not capable of being closed, and were intended to restrain only a part of the flow. General Chittenden apparently dismissed this type of reservoir on the authority of this article.

When exactly this same type of flood control was proposed for the Miami Valley in Ohio, General Chittenden was asked to review the plans, as he was the ranking authority in opposition to reservoir flood control. After long and intensive study, he approved the Miami Conservancy program unqualifiedly, and became one of its strongest public advocates. Either the proposed French dams were not properly designed, or their limitations did not apply to certain American conditions.

Seventy-five years ago European engineers did not think on the modern scale. Methods of earth dam construction that now are fully tested were then unknown, and some sites then rejected would now be looked upon as exceptionally desirable. Projects costing $10 000 000 or $20 000 000 are referred to as too vast for accomplishment by the French Government. It was the day of small undertakings.

Later, in the paper by General Chittenden, occur these statements:

"The best reservoir site is a natural lake. * * * A natural lake, wholly uncontrolled at its outlet, may have a more effective control of the outflow than an artificial reservoir of equal superficial area when full, although of far greater capacity between high and low water. * * * if the artificial reservoir has reached the limit of its allowable filling, the outflow must be made equal to the inflow. If this limit is reached before or at the time of maximum run-off, then a quantity equal to this run-off must be let out of the reservoir. This contingency can never happen in a natural lake."

At the time General (then Colonel) Chittenden read this paper, very little study had been made of flood-control reservoirs. It is now known that the limitations of artificial regulation, which he mentions as inherent, simply represent imperfect design. General Chittenden himself fully recognized this in his studies of the Miami River problem. No natural lake is ever so good as a medium of flood control as the same body of water would be with a properly designed outlet.

Further on, in the same paper, he states:

"Every reservoir built for the purpose of flood protection alone would mean the dedication of so much land to a condition of permanent overflow in order that three or four times as much might be redeemed from occasional overflow. * * * The cost, coupled with the loss of so much land to industrial uses, would be far greater than that of levees or other methods of flood protection. * * * The construction of reservoirs for flood protection is not, therefore, to be expected, except where the reservoirs are to serve some other purpose as well."

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* *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 287.

† *Engineering News*, Vol. 25, p. 258, March 14, 1891.

Here, again, the design of flood-control dams which General Chittenden had before him for criticism was inadequate and inefficient, and is now superseded by more suitable design.

Some of the faults and limitations pointed out by General Chittenden in the design and operation of the Mississippi reservoirs in Minnesota are not inherent, as he implies, but are due to improper design or operation, or to conflict of interest resulting from an effort to make them serve more than one purpose. In his discussion of proposed reservoirs on the Sacramento and Kaw Rivers, and in his quotations from other sources, incorrect design is assumed, which largely vitiates his argument.

At the time the Miami Conservancy flood-control dams were being planned, only two flood-control dams could be found in the United States. One of these controlled a local brook, and the design of the other was so primitive and inefficient that it must largely fail of its purpose. Such was the inadequate data on which General Chittenden and his contemporaries had to work.

That General Chittenden had been unconsciously indoctrinated by the standard creed of the Mississippi River Commission on flood control, and that he only gradually freed himself, is indicated by the following comment by him when he was criticizing the traditional position of the Commission on the matter of outlets: "The writer feels the more freedom in criticizing the Committee's treatment of this matter because he is at the same time criticizing his own previous utterances on the same subject."*

After his long study of the Miami Conservancy plans General Chittenden approved the methods he formerly had condemned, and his last professional effort was a contribution to the *Transactions* of the Society pleading for an open mind on the subject.

At the beginning of the Miami Conservancy studies the writer had the same convictions as those of the Mississippi River Commission. He was certain, on the authority of the Mississippi River Commission and the Army Engineers, that reservoirs were not feasible for flood control.

On the Miami Conservancy project the possibility of reservoirs was examined simply to carry out the sound engineering policy of examining into every possibility, no matter how remote. The writer had very vigorously expressed his disbelief in reservoir flood control to the Flood Prevention Committee at Dayton. When the evidence forced him to change his views, no one was more surprised than he. At the beginning of that project not more than two or three men in a large engineering force believed in the possibility of flood control by dams. At the end every one was convinced.

Even then the writer's faith in the doctrines of the Mississippi River Commission was only slightly shaken; reservoir control could be good only on small streams. Only recently he wrote an article on the Mississippi,† in which he criticized the Mississippi River Commission, but assumed it to be substantially right in the matter of reservoirs.

In the preparation of the present paper, however, he has hunted assiduously for those deep researches and critical studies so often referred to by the Mis-

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1269.

† *Atlantic Monthly*, November, 1927.

sissippi River Commission and by the Army Engineers. To his astonishment, he finds they never have been made. Repeatedly there is positive assertion and the attitude of great authority and always a complete lack of any adequate engineering analysis.

Following back to original sources one finds the obsolete French article before referred to, and the report of the Engineering Board of 1874 which led to the establishment of the Mississippi River Commission. In this report, so often referred to as authoritative, may be found expressed a fundamental misconception of the proper operation of flood-control dams, followed by the following conclusive remarks:*

"The question of absolute practicability could only be decided by a series of extensive and elaborate surveys, for which neither funds nor time were available, nor in the opinion of this Commission are they needed. Here, as elsewhere in the valleys, this plan, as an efficient means of restraining the floods of the Mississippi, is chimerical."

(This was the Board which estimated the total cost of complete protection by levees at $46 000 000.)† A discussion by the late W. Milnor Roberts,‡ M. Am. Soc. C. E., to prove that reservoirs are not feasible is repeatedly referred to, and is mentioned as disposing once for all of the reservoir theory. Yet when we examine this article we find no discussion and no conception of modern flood-control reservoirs. His discussion is obsolete. Coming down from that early date to recent years we find one of the last affirmations of the attitude of the Mississippi River Commission in the report of the Special Committee on Floods and Flood Prevention of the Society. The Chairman of this Committee was President of the Mississippi River Commission, and the report presents the orthodox Mississippi River Commission attitude, with slight concessions and some caution. In deciding against reservoirs and in favor of levees this Committee states that for rivers like the Mississippi and the Colorado, levees "afford the only sure means of flood control."§ These are the only two streams particularly mentioned, and yet, to-day, only ten years later, engineers familiar with the situation generally recognize that to a considerable degree reservoirs do offer a sure means of flood control on the Colorado.

William Kelly, Colonel, Corps of Engineers, U. S. A., M. Am. Soc. C. E., in 1924, definitely recommended reservoir flood control for the Colorado River. He states that‖: "Storage for flood control is justified to a quantity that will reduce the maximum discharge for any year of record to 75 000 sec-ft."

The writer has not presented simply a few weak points from the arguments of the Mississippi River Commission. He has presented practically all the arguments it has offered. So far as work done before 1927 is concerned, the adequate scientific study of reservoirs, so often referred to by members of the Commission, in fact is entirely a myth.

In view of the great variety of topographic and other conditions over the drainage area, it seems probable that an exhaustive study may show this

* Report of Special Board of Engineers of 1874–75, Pt. I, p. 541.

† *Loc. cit.*, Pt. I, p. 563.

‡ "Practical Views on the Improvement of the Ohio River," *Journal*, Franklin Soc., 1857.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1228.

‖ *Loc. cit.*, Vol. LXXXVIII (1925), p. 323.

method of flood control to have some proper part, if only a very limited part, in a well-considered general plan. Dependable conclusions must come from thorough study of particular cases.

In discussing suggestions that have been made by the advocates of flood control on the Mississippi by means of reservoirs, the writer finds his attitude is not that of unqualified approval. There have been many suggestions for such control, but not many of them have been worked out in detail.

One of the most carefully planned projects is that of a proposed system of reservoirs above Pittsburgh, Pa. The writer has not kept informed of that situation since the publication of the voluminous report of the Pittsburgh Flood Commission about 1910. An examination of that report, together with numerous conferences with some of those most directly responsible for its preparation, led him to the opinion that in this case the difficulties in the way of combining storage for power, flood control, and navigation in the same space would be difficult to overcome, because of inability to foretell rainfall and run-off conditions, and from the impossibility of operating gates to harmonize the demands of flood control and power development.

The difficulty of knowing when to store water for power and when to reserve storage capacity for floods is illustrated in the case of the Miami River in Ohio. In March, 1913, there was an average rainfall of 10.4 in. over the Miami water-shed above Dayton; in March, 1915, only 1.4 in. Floods come too suddenly to provide storage capacity for them by emptying reservoirs after a heavy rain has fallen, and under such variable climatic conditions it would be impossible to tell when to keep the reservoirs full for power development, and when to keep them empty for flood control. There are many cases where the two purposes can be combined, such as the proposed Boulder Canyon Dam, but popular optimism in the matter is not well grounded.

One of the chief resources of the Mississippi River Commission in its battle against reservoirs has been this weakness with which the case in favor of reservoirs has been advocated, especially by other departments of the National Government. The case in favor of reservoir flood control sometimes has been presented with an irresponsible optimism, with an unconcern for the absence of dependable information, and with a disregard of difficulties, that is disconcerting. If one is planning a military campaign it is good strategy to exploit the enemy's weakness to the utmost. Where the aim is not to overcome an opponent, but to find the truth, a different course is indicated. One of the most valuable proposals the writer ever received was from an engineer who was so visionary and impractical that he could by no means be trusted with its execution. The Mississippi River Commission might profit in the same manner from the suggestions of its adversaries.

The matter of reservoir control is a technical one and cannot be mastered except by the same kind of skillful, thorough-going, and definitely directed study that is necessary for the mastery of any difficult technical problem. No happy inspiration, no casual sallies into the field, like those of enthusiastic advocates of reservoir flood control, and no blind loyalty to a traditional cause, like that of the Mississippi River Commission, can bring dependable results. The Mississippi River Commission seems to have trusted to the advocates of

reservoir flood control to make a sufficient presentation of the case for reservoirs. The Army and the Commission have practically controlled the purse strings for the Mississippi. An adequate analysis of the problem would require years of time and very considerable funds for the accumulation and analysis of data, and the Army Engineers or the Mississippi River Commission have been the only agencies with power to secure the necessary funds.

The Commission is to be criticized, not for failure to understand the issue quickly, but for ignoring it, and for letting the years of its stewardship pass without a definitive study of the entire Mississippi problem.

In case there should be opportunities for Mississippi flood control by means of reservoirs, where might the sites be found? In the writer's opinion they most probably would occur just above the mouths of the main tributaries, such as the Yazoo, the Red, and the Arkansas Rivers, where present back-water conditions probably should be continued, and on these main tributaries and their branches, probably not more than 200 or 300 miles from the Mississippi. The greatest benefit would be expected from a few great projects, rather than from many small ones.

The writer believes that there may be opportunity for very considerable storage, amounting to many millions of acre-feet, on the Lower Ohio itself, and near the outlets of its main tributaries, as well as farther toward the head-waters. If properly designed with low-water outlets, high dams could be built without undue interference with navigation.

The St. Francis and Yazoo Basins, which are among the very best of the Mississippi bottom-lands, and perhaps other parts of the overflow area along the river, must eventually receive protection from local flood water by dams in the hills. No one of the rivers entering these basins from the hills has a capacity equal to 5% of the extreme flood flow. The dams which are necessary for flood control on these streams may have a limited but significant effect on the main river.

Retarding basins of large capacity probably are feasible on the Red River, and on other large tributaries. No human being knows what the possibilities are, because the necessary investigations never have been made.

If we look the facts in the face, we see a heroic job ahead, one that will involve hundreds of millions of dollars of expense. Whatever type of protection is secured must be on an unprecedented scale. Under these conditions the whole aspect of the problem is changed. From time to time the problem of flood control on the Mississippi should be examined *de novo,* as if nothing were known about it, to discover how the new scale of thinking will affect the plans. The writer believes this to be desirable, regardless of the skill and standing of the men who have made previous studies, because the state of professional knowledge and the issues to be met change so greatly that old decisions cease to be authoritative. Dams on the Ohio have been dismissed as involving too many complications. The difficulties that formerly seemed insurmountable may now be only necessary incidents of the best plan.

What would the completion of Mississippi flood control by dams and retarding basins be worth as compared with completion by further raising the

levees? Only an exhaustive study can give an adequate answer, but a few items can be suggested. Assume that the present levee system is retained, and is supplemented by dams which will hold the river within its natural banks except during years of extreme run-off, at which time the levees will come into play.

First, there are reported to be about 3 000 000 acres along the Mississippi River that are not protected by levees. The writer has not personally checked this estimate. This is recognized as the most desirable agricultural land along the river, as for the most part it is sandy loam, and not the sticky gumbo of the lower land in the basins. To reclaim an equal amount of swamp land in the Yazoo, Arkansas, or Red River Basins would cost not less than $20 per acre. This land along the river-front is high, and needs little drainage. Assuming that the land between the river and the levees should be submerged in great floods once in 10 or 20 years, such flooding would tend to renew its fertility, and it would not lose its value. With the degree of protection just mentioned, this land along the river-front would be worth probably $50 per acre more than it is now, an increase in value of $150 000 000.

High water causes most of the caving of river banks. The reduction of floods would greatly decrease this caving, and might save levee replacements and revetment work ultimately running into hundreds of millions of dollars. A credit of $50 000 000 for this item is very conservative.

The completion of interior drainage and local flood control of the St. Francis, Yazoo, and other basins, except by means of dams and retarding basins on their own water-sheds, will cost more than $250 000 000, and possibly $500 000 000. Since the Morgan Engineering Company has made plans or superintended the reclamation of more than 1 000 000 acres of this land, the writer feels that he can make an estimate that is rough, but not without value. Flood control by dams and retarding basins on the streams entering these basins, in addition to value in reclaiming the low lands of the basins themselves, would help control the Mississippi, and might properly be credited with $50 000 000.

As to the cost of complete Mississippi flood control by levees, only a guess can be made. The estimates of the Mississippi River Commission have been totally inadequate, and there are no others. We now hear rumors that the new report of the Mississippi River Commission will indicate a cost of completion of $500 000 000. Perhaps, for the first time in the history of the Commission, the estimate will be adequate. Assume that completion of control by dams and retarding basins would save further levee construction, and so may be credited with that amount, in this comparison of costs and values.

Control by reservoirs would give protection to cities on the Ohio River and perhaps on other streams. In some cases, as at Cincinnati, Ohio, control by any other means than reservoirs seems remote or impossible. This item of local protection can safely be credited with $50 000 000.

Some advocates of reservoirs have gone far wrong in assuming that there is little difficulty in combining diverse uses, such as power development, flood control, and navigation. The writer has been insistent in drawing

attention to the limitations of such combinations of use, yet those difficulties are not absolute, but are a matter of degree. Some degree of combined use is possible and desirable. Only a wild guess can be made as to the value of such a system of dams for other purposes, but an item of $50 000 000 would seem to be well within safe limits.

Finally, there would be many incidental benefits, some of which would not be immediately apparent. The higher the water rises behind levees on the Lower Mississippi the farther will it back up the tributaries, flooding the lands and interfering with drainage. Reservoir control would very greatly reduce this damage. Probably three-fourths of the silt and sand carried by the river is caved from the banks, carried a short distance, and deposited again. A more uniform flow, by reducing caving, would relieve the river of this local burden of silt, and make its silt-carrying capacity available for the permanent deepening of its bed. Much land along tributary streams and below the upper dams would be saved from frequent flood damages. The construction of dams might reduce the need for outlets or spillways. Omitting such least measurable values of reservoir control there is a value of more than $750 000 000 to offset the cost of further levee construction. It is probable that the value would reach $1 000 000 000.

In the practice of engineering there are two fundamental processes, both of them invaluable, which should supplement, and not compete with, each other. One method is that of perfecting and refining existing methods; of letting future policies grow gradually out of past experience. The other method is that of fundamental scientific analysis and engineering design.

The breeder of race horses followed the former method and with a remarkable degree of skill, but it required fundamental engineering design and a new synthesis of elements to create the automobile. The teacher of elocution may help the orator to reach a larger audience, and may use scientific principles in the details of his work, but it required a process of far more fundamental analysis and synthesis to produce the telephone and the radio.

The employees of the Mississippi River Commission, and the Army Engineers working under their direction, sometimes have used excellent scientific methods in their work, but in its larger aspects the whole policy has been that of the practical rule-of-thumb man, and has almost entirely lacked thoroughgoing scientific analysis of the larger engineering problems. The writer believes the major policies which have governed flood control on the Mississippi River justify the following remark of John R. Freeman, Past-President, Am. Soc. C. E.,

"That the steamboat pilot on the river and the scientist in his laboratory each has certain advantage in his point of view, and that much of the river training work in America has been of the quality that might be expected to be produced by a committee of steamboat pilots, without special training in exact science."

If the problem must of necessity be assigned to either type alone, it should be to the man of practical experience, but in the present instance the country sorely needs a combination of both types of service. A member of the Mississippi River Commission recently complained of the criticism that "no

adequate scientific study has been made of this important problem."* A study of the basis of the conviction of the Mississippi River Commission against reservoirs has completely confirmed the writer in this opinion.

The improvement of the Mississippi River is under the control of the Mississippi River Commission, which includes both Army and civilian engineers. The Army Engineers assigned to district offices carry out the policies of the Commission, and are not expected to originate policies. As the experience and the duties of Army Engineers are largely concerned with administration, rather than with technical engineering studies, and as these assignments to posts on the Mississippi River are usually for short periods, they do not have time nor opportunity to become qualified as experts on Mississippi River control. Under the circumstances they generally reflect the attitude of the Commission. The civilian assistants in the district offices have worked under somewhat similar limitations, except as to tenure of assignments.

The orthodox attitude of the Mississippi River Commission is the traditional attitude of the engineers along the river. In general, the influence of the Army Engineers has been to raise the standards of the Commission. It is generally believed that such spirit of research and inquiry as has existed in the Commission has largely been due to the Army Engineers. It is generally understood, also, that they have prevented very serious blunders by the Commission, such as the closing of Old River.

It is impossible to separate the history of the Mississippi from the work of the Army Engineers, but it should be realized that in their short assignments on the river and in inheriting the old standards of engineering, they have largely been the victims of circumstance. The present administration of the river is not conducive to the type of engineering analysis that is imperative.

For nearly fifty years the Mississippi River Commission has held to an arbitrary dogma of "levees only" as the final solution of this great problem. Resting on the infallibility of that doctrine, it has discouraged a thorough scientific analysis of the problem as a whole. To-day, with the estimates of half a century discredited by events, and with the public ready to furnish the resources for protection, it finds the precious years largely wasted, so far as any comprehensive research on the problem as a whole is concerned.

When a good soldier finds himself in a hard position he determines to "fight it out on that line if it takes all summer." He knows that "many a battle has been won by a bad general, but that no battle ever was won by a debating society." Let us be thankful for the hard fighting quality of our Army officers. But when a scientist finds that he has taken a questionable position his method is to examine that position to discover whether it is justifiable. There are situations in which the attitude of the scientist is no less necessary to the nation than that of the soldier.

If there should be one chance in ten that reservoirs can play an economical part in the control of the Mississippi, then it is unscientific and unprofessional to adopt an engineering plan for the control of the great river without

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1480.

having thoroughly examined into that possibility. The writer believes that there is in fact a much greater chance that control by dams and retarding basins has an economical, effective, significant, and necessary place in such a policy, even if it be a small place. No more definite opinion is justified in view of the limited data available.

During the preparation of this paper he has become convinced that the examination which is necessary to determine those possibilities never has been made. He strongly believes that for the National Congress to be committed to a permanent policy with reference to flood control on the Mississippi, without such inquiry, will be a mistake of historic magnitude.

The writer heard with much interest the paper of Colonel Kelly.* General Jadwin, Chief of Engineers, U. S. Army, and his associates, are attacking the problem of the Mississippi with an intelligence and a comprehensive view that are promising, and command the admiration of the profession. Had that outlook characterized the study of the river for thirty years past, the present dilemma as to plans would not exist.

There doubtless will be great pressure upon the Congress about to convene to make appropriations for reservoirs on the distant head-waters of the Mississippi in the guise of helping Mississippi flood control. The writer thoroughly agrees with Colonel Kelly that no reservoirs on the distant head-waters of tributaries to the Mississippi can have any appreciable effect on Mississippi floods. He doubts whether there is a single irrigation project in the United States on which the building of reservoirs would have any markedly favorable effect on the lower river. The Army Engineers should have the full support of the Engineering Profession in their efforts to prevent the misuse of funds through appropriations for unsound reservoir projects. This statement is a necessary introduction to any criticism of the recent study of reservoirs described by Colonel Kelly.

A criticism of that study, however, seems necessary. If the analysis of the entire Mississippi problem, so intelligently outlined and undertaken last June by the Chief of Engineers and his associates, could have been spread over a number of years, the country would be fortunate. To endeavor to crowd into six months the work of ten years is not productive of dependable results. The writer believes this is true of the recent study of reservoirs described by Colonel Kelly, for the following reasons.

Until recent years there has been little understanding in America of the principles of design and operation of flood-control dams. In the search by Army Engineers some years ago for flood-control reservoir sites in Ohio, not one of the sites used on the Miami Conservancy project was discovered. Every one of the most significant possibilities in the State was overlooked. In the development of the Miami Conservancy plans the writer found that widely experienced and competent hydraulic and power development engineers, who had not thought in terms of flood control, often had very erroneous conceptions of the problem. No hasty examination would have developed the reservoir possibilities of that project. The actual necessities and possi-

* See p. 2519.

bilities could be discovered only by a process of thorough and fundamental analysis and design.

The control of the Mississippi by reservoirs or by any other means will require engineering imagination and creative design on an unprecedented scale, and only persons especially prepared, and having a clear understanding of the problem as a whole, would have a concept of the nature and functions of such reservoirs, or would recognize the existence of possible sites. The writer does not believe it is possible to send instructions to a miscellaneous group of division engineers, some of them burdened with flood-relief work, whose past duties have been largely administrative, few of whom have had any experience with flood-control reservoirs, or have given them any thought, and to get hastily prepared reports of very much value.

The writer understands that instructions for the examination of reservoir sites were sent to the various division engineers about July 1, that preliminary reports were requested in about two weeks, and final reports about six weeks later, or about September 1. More than 400 000 sq. miles of territory had to be reported on in that time, only a minor part of which was covered with topographic maps. The writer believes that under these conditions there was small chance that the significant sites would be recognized and properly appraised. To assume that errors would cancel each other out is somewhat like giving a mathematical problem to several men who do not understand it, and taking the average of their answers as correct.

The writer believes that the rule adopted, of considering that reservoir gates would be closed on February 1 for three months, is an arbitrary and improper assumption, made necessary by the great haste in which the work had to be done, and that this rule alone would largely prevent the recognition of favorable sites.

He believes that this study will be worse than useless if it leaves the impression that an adequate investigation has been made, and if the possibility of a material degree of control by reservoirs is thereby dismissed without adequate examination. After hearing Colonel Kelly's paper, the writer still is of the opinion that an adequate study of this problem still remains to be made.

In his paper* Colonel Townsend, former President of the Mississippi River Commission, stated that in the further raising of the levees the cost would increase approximately as the square or the cube of the height. The last foot of raise is far the most expensive. Colonel Kelly, on the other hand, correctly states that if reservoir control is sought, the top foot of water is most easily held back, because the most economical reservoir sites are first chosen. Thus, the work which can be done most economically by reservoirs is that which is most expensive if accomplished by levees. The writer believes it is very probable that at some point, to be determined only by very thoroughgoing analysis, the curves of economy will cross, and the balance will be in favor of reservoirs.

The confusion of counsel and the conflict of opinion running all through the present discussion of this subject is not due to absence of engineering

* See p. 2461.

intelligence, but to a lack of the information necessary for sound judgment. If the Mississippi River Commission for the past thirty years had made the necessary studies, these controversies would already have been settled by the facts.

On the Miami Conservancy project there were at first the same conflicts of opinion, but the technical studies of the district settled nearly all mooted questions, and the technical design adopted has been generally approved by American engineers. Similar unity concerning the Mississippi problem will follow a proper study of the facts. No other policy than long-continued, thorough-going, comprehensive analysis will bring that unity. The country should not be committed to a far-reaching flood-control policy without such study. At some time, the fact of lack of information must be acknowledged, and this policy of thorough-going analysis adopted. It will then take years of such study to arrive at a dependable decision.

In conclusion, the writer would emphasize the fact that he is not an advocate of reservoir control for the Mississippi. He does not know to what extent such control is feasible; but he does advocate a deliberate and conclusive study of the subject, which, he believes, never has been made.

LEVEES AS A MEANS OF FLOOD CONTROL FOR THE MISSISSIPPI RIVER

By J. F. Coleman,* M. Am. Soc. C. E.

The control of the floods of the Mississippi River is truly a great engineering problem. It involves uncertainties and difficulties to a greater extent than did the Panama Canal, or any other great work of which the writer has knowledge.

Control of these floods on a basis of any method which has been suggested would involve the expenditure of great sums of money which have not hitherto been available. For this reason the task of those who have been responsible has been to expend such funds as could be obtained in such manner as would afford the most protection. In the main, they have performed this task creditably. Political expediency and sectional jealousy have played their parts in impeding the appropriation of sufficient funds with which to accomplish the task.

By reason of the vast territory and large population whose interests are so greatly involved, the problem has received the attention of many men, both technical and non-technical, some at least of whom have not hesitated to proclaim full and final solution with little or none of the fundamental data on which a true solution must be based.

There can be no better forum than the Society before which to discuss this great engineering problem. The subject has been before it on past occasions and no doubt will be again until the true solution is found. The problem is far more complex than appears upon superficial examination and it is not to be expected that the ultimate solution will be attained without research, study, and analysis, such as would be applied to other engineering problems. It is unfortunately true that an enormous amount of misinformation is extant, which makes it necessary for the student to check carefully a great many of the data which he will assemble before reposing too much confidence therein.

As in the study of other problems in river hydraulics, one is soon brought to a realization of the vagaries and eccentricities of water flowing in open channels of the character of river channels, such, for example, that a given cross-sectional area and gauge reading does not always produce the same velocity and discharge; or that a given gauge reading at one point on the river does not always produce another given gauge reading at another point lower down; or that the flood-plane of the river along any given part of its length is a broken line, and that there is frequently an appreciable variation between the line showing the flood-plane at one time and that of another time when the river stage is approximately the same, etc.

The various formulas for the flow of water in open channels, which have served useful purposes in smaller streams and in canals, often serve to mislead when applied to so large a stream as the Mississippi River.

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Necessarily some of the factors to be dealt with may not be definitely known, and for practical purposes must be assumed. The most important of these is the volume of water which must be provided for. The volume which has flowed down the main river channel during each of what may be called the high-water years for the last thirty or forty years is known with close approximation; but there has never been a time when all the tributaries of the main river have been in maximum freshet simultaneously, or rather at such time as to deliver their respective flows into the Mississippi River in synchronism with the wave crest of the flood in that river.

It is probable that there never will be such a time. It also seems probable that the flood of 1927 will never be exceeded. On the other hand, it is possible for the river to be called on to handle a greater flood than that of 1927. This flood problem is a very vital one to the residents and property owners of the Mississippi River Valley and to those of the valleys of many of its tributaries. Perhaps there has never before been a time when it has received the attention of as many engineers as are now concentrated upon it; and it is probably true that a greater amount of information is now, or soon will be, available than has heretofore existed.

It is to be regretted that the date of this discussion could not have been postponed for about six months, by which time all the discharge observations, surveys, etc., which have been and are being made by the Mississippi River Commission and other authorities, might have been available for study and analysis.

All the methods proposed or suggested for controlling the floods of the Mississippi River include levees; and all of them also include revetment for the control of caving banks. The writer is among those who believe that levees and bank revetment, without the aid of reforestation, contour plowing, reservoirs, outlets, or spillways, will most surely provide the necessary control for the floods of the Mississippi River. This river, it will be remembered, is an alluvial stream flowing in a bed of its own creation. Its burden of silt is heavy and its need for energy with which to transport its silt-laden waters is great. The channel, as it now exists, is admittedly capable of carrying a volume of about 1 000 000 sec-ft. at a bank-full stage from Cairo, Ill., to the Gulf of Mexico. The maximum discharges recorded prior to 1927 were, as follows:

Place.	Year.	Second-feet.
Columbus, Ky.	1912	2 015 213
Helena, Ark.	1912	2 040 660
Arkansas City, Ark.	1912	2 006 601
Vicksburg, Miss.	1922	1 825 555
Red River Landing, La.	1912	1 499 402
New Orleans, La.	1922	1 358 311

The flood of 1927 is said to have been the greatest on record. There is, however, a divergence of opinion as to its volume. Estimates range from 2 000 000 sec-ft. to as much as 3 250 000 sec-ft. Whatever may have been the real volume it would surely have been more than 2 000 000 sec-ft. if entirely

confined within levees, which would have been more than double the capacity of the bank-full river. It is apparent, therefore, that the channel without levees is not large enough to carry the burden in time of freshets and must be enlarged.

A part of this necessary enlargement is taken care of by the leveed section above the bank-full stage. More may be provided by enlargement of the section below that stage.

The "Levees Only", or "Confinement", plan is based on the theory that, in an alluvial bed or channel, the energy and power required for channel enlargement may best be obtained by so confining the flowing water that its velocity will be increased and its scouring capacity will be put to the useful purpose of enlarging the channel.

According to the same theory, the removal or diversion of any part of the waters from the main stream reduces the velocity (and, consequently, the energy) of the main stream below the point of divergence and results in deposits of silt and the reduction of cross-section of the main channel, and hence reduces its carrying capacity; or, to state the proposition in the language of the first report (1880) of the Mississippi River Commission:

"If the normal volume of water in a silt bearing stream flowing in a bed of its own formation be permanently increased, there will result increase of velocity and consequently of the erosive and silt bearing power; an increase of depth, if the banks are held, and an ultimate lowering of the surface slope; and conversely if the normal flow be decreased in volume, there will ensue a decrease in velocity, silt transporting power, and mean sectional area, and an ultimate raising of the surface slope."

Let us examine how this theory fits the records of such streams of this character as are available.

In 1913, the late J. A. Ockerson, Past-President, Am. Soc. C. E., compiled experiences and conclusions of eminent authorities as to the utility of outlets,* from which the following are quoted:

"(1) Result of the diversion of the Adige into the Castagnaro, Italy.—Elevation of flood level by divided flow and conversely a lowering of flood level when waters were restored to one channel.

"M. R. C. 1890, p. 3106:—'About A. D. 1438 the Adige broke its levees and poured its waters south into the Castagnaro and Canal Bianco, which then formed a drainage stream parallel to the Po. In 1545 the break had so increased, that two-thirds of the low-water flow of the Adige, and three-fourths of the high-water flow, went through it. A low dam was built across the Castagnaro to check the flow into it, and both rivers raised their beds. In 1678 a new dam was built, as the old one was then buried in the deposit. The bed still rose. In 1791 a masonry dam, 39 feet high, with many arch ways through it, to allow floods to pass, was built across the Castagnaro. The bed continued to rise and the floods on the Adige were so high that in 1838 the Castagnaro was permanently closed. In the six years following the closure the floods in the Adige fell, and the more markedly the nearer the point considered was to the Castagnaro.'

"(2) Extracts from work of Frizi, an eminent Italian Engineer, published in 1762.

"Of the Rhine he says: 'The great multiplicity of channels, though productive of very great advantages to the navigation and commerce of Holland,

* "Outlets for Reducing Flood Heights", p. 14, pub. by Mississippi River Comm., 1914.

draws after it very fatal consequences. The waters divided into so many branches lost the rapidity and strength which are required to sustain and push forward those heterogenous substances which they transport. The constant rising of the bottom renders the drainage of the waters from the fields more difficult, increases the expense of the necessary embankments, and always augments the damage which these extensive lowlands suffer when the dikes break and threaten the whole country in ruin.'

"To remedy that trouble it was proposed in 1754 to provide still more channels and sluice ways. Gennette, however, maintained that this would not diminish the height of floods; that all of the several channels should be abandoned save one through which the waters should be carried to the sea by the shortest practicable route, and his advice was followed with satisfactory results.

"The effect of adding to or subtracting from the volume of a river is described as follows: 'We read in the collection of observations for 1728 that, having made the experiment of placing a mark (gage) in the Panaro (river) and of letting in and afterwards withdrawing the waters of the great drain of Burana, they observed in the Panaro no sensible rise in the first instance, nor any visible decrease in the second. These three facts have been particularly attested by Eustice Manfredi, whose testimony is worth that of all others. No objections can be made to these facts, for it cannot be said that the quantity of water in the affluent bore no sensible proportion to that of the recipient stream, nor that the sections of the recipient were not effective; neither can the variability of these very sections be attributed to any other cause than an increase of velocity in the united waters, proportioned to the increased quantity of water itself.'

" '* * * In diverting from the principal channel a considerable volume of water, that which is left behind is not visibly diminished either in height or in breadth.'

"Other early observations on the Po confirm this principle.

" 'It is a hydrostatic paradox, commonly taught by Italian authors, and uniformly confirmed by experience, that you do not diminish the height of the waters in great floods by lessening the quantity of water.'

"Father Castelli, in the 13th Corollary of his first book on Running Waters, disapproved of the division formerly made of the Po at Buondens and which was afterwards abandoned. (In 1838).

"Guglielmini confirmed the opinion as to the little utility of spillways for reducing flood heights in streams.

"Eustice Manfredi proved the futility and danger of spillways proposed for the right bank of the Serchid River. Experience in like manner demonstrated the futility of a cut made in the right bank of the Arno to prevent the flooding of the City of Pisa. This cut was made in 1740 but no perceptible diminution of flood height at Pisa occurred. The cut was made anew in 1761 during a great flood but the waters continued to rise so high that the people 'could not be persuaded that the cut had been made.' In fact, the greatest flood within the memory of man was recorded.

"Cornelius Meyer, a celebrated Dutch engineer, disapproved of this method of dissipating the flood, and instead proposed confining the water to a single channel with ample embankments to carry the flood waters safely to the sea.

" 'The canal made by order of the Emperor Nerva to draw off the superfluous waters of the Tiber at the times of its great freshets, did not contribute in the smallest degree to prevent inundations.'

"One may also inspect the discourse of the celebrated Lorgna on the inundations of the Adige, which sufficiently proves that all the derivations made in that river have only produced a heightening of its bed, and thereby rendered its floods more dangerous.

"The following rules, as set forth by Guglielmini, govern the regimen of alluvial streams:

" 'The greater the ordinary body of water in a river the less will be the slope of its bed.

" 'The greater the quantity of water which a river carries, the less will be its fall; and the greater the force of the stream, the less will be the slope of the bed.

" 'These two rules resolve themselves into a single rule, viz., That the slope of the bottom in rivers will diminish in the same proportion in which the body of water is increased.' "

In the Annual Report of the Mississippi River Commission for 1881, page 128, occurs a statement to the effect that levee building on Red River below Shreveport, La., began about 1860; and that the bed of the river has been depressed and "lands have not been inundated for years."

The writer has not been able to obtain actual records of the surveys made of Red River within the past thirty-five years, and, hence, is unable to give exact figures. Mr. Gervais Lombard, a member of the Board of Louisiana State Engineers, who has been in charge of levee work on Red River for many years, states that below Shreveport the size of the channel has increased about 300%; that the bed of the river is generally lower in elevation; and that a flood of given volume is carried out of that river now with a lower flood-plane than was possible prior to the completion of the levee system.

The Upper Yazoo River Levee District has enjoyed absolute protection from floods for a number of years. That District has been fortunate in having adequate funds with which to build superior permanent levees and auxiliary works. The levees have a free-board of about 5 ft.; are heavier than the standard; have a hard surface roadway on the banquette; and are systematically and effectively maintained by regularly employed section gangs. The writer is not able to furnish information in specific terms as to the size of the existing channel by comparison with the channel at the time of the completion of the levee system. It is understood that the channel has enlarged itself.

On the Mississippi River many opportunities have been lost for gathering and recording valuable data in precise terms. This has been due, no doubt, to a lack of sufficient funds, and perhaps, in part, to a failure to appreciate the value of such data.

Many outlets and spillways have been closed; many crevasses have occurred, some of which remained open for several years, but all of which were ultimately closed. Neither in the case of the closed outlets and spillways, nor in that of the crevasses, has there been available accurate surveys of the river before and after. It is known that just below each outlet and spillway the river contained shoals and sand-bars which were obliterated after the closure. It is also known that below each crevasse the river has been reduced in section and that, when the crevasse remained open over a period of years, this reduction has been progressive; also that upon the closure of the crevasse the re-enlargement of the cross-section immediately began.

For lack of actual surveys, it is not possible to furnish precise figures of the reduction or of the enlargement resulting from any crevasse and its

closure. The only outlet which still remains open (other than the yet unclosed crevasses of 1927) is Old River (just above Red River Landing), through which large volumes of water leave the Mississippi River in high-water periods and pass through the Atchafalaya River to the Gulf. Surveys have been made of the Mississippi River in the vicinity of this outlet from time to time over a period of years.

Cross-sectional areas based on "low water" at Torras Landing and at Red River Landing, respectively, about ½ mile and 1 mile below the outlet, are given approximately in Table 28.

TABLE 28.—CROSS-SECTIONAL AREAS, IN SQUARE FEET.

Place.	Year.				
	1884.	1908.	1918.	1922.	1924.
Torras Landing..................	61 300	21 150	21 300	19 900	23 600
Red River Landing.............	73 800	30 500	29 850	36 200	30 500

For some years past it has been necessary to dredge the channel in low-water seasons in the interests of navigation, and it is not known to the writer whether or not this dredging may have had the effect of maintaining a fairly constant sectional area from 1908 to 1924. Later surveys are not yet available. It is noteworthy, however, that the cross-sectional areas from 1908 to 1924 are only about 35 to 40% of the cross-sectional areas of 1884.

It seems safe to assert that a crevasse, a spillway, or an outlet will surely result in a progressive reduction in the main channel a short distance below the said crevasse, spillway, or outlet, and that this progressive reduction of channel will continue as long as the diversion of water from the main stream continues. There is also much evidence to support the belief that the crevasses and spillways have a tendency to silt up themselves gradually and, therefore, to grow less and less efficient as the years go by, although the evidence is not in the form of actual surveys.

If technically and practically the "Confinement" theory is right, as the writer believes it to be, it will be necessary either to close the mouth of Old River and confine all the waters of the Mississippi River within the channel of that stream, or to close the Mississippi River just below the mouth of Old River and divert the waters to the Gulf of Mexico through the Atchafalaya River. The latter route is shorter by about 150 miles, but the volume to be provided for would be greater than in the former case by the addition of waters from Red River and its tributaries.

It is not the purpose of this paper to select the channel, or to enter into questions of details of levee construction or bank protection. It seems to the writer that the first step to take in a study of this great problem is to determine, with as much certainty as possible, whether it is better to concentrate the flood waters in one channel, or split these waters into two or more channels by maintaining the Old River outlet and by creating other outlets

and spillways. When the broad general principle is settled, it will be time enough to attempt the determinations of other points.

The unquestioned fact that the stream of large sectional area will attain greater velocities and consequently greater discharge capacities with a given hydraulic slope than would be possible with smaller streams of equal total sectional area is a strong point in favor of the "Confinement" theory. The increased velocities resulting from confinement of the flood waters and the increase in silt-bearing capacity and in scouring capacity, which follow such increase in velocity, hold forth promise of the enlargement of the channel by the erosive forces of the contained waters, and an enlarged channel is needed to carry the excess burden of freshet waters.

The knowledge that the diversion of waters by outlet, spillway, or crevasse will surely check velocities and cause deposits of silt, and hence will reduce the cross-section of the main channel and foreshadows ultimate reduction of carrying capacity as a natural sequence.

It cannot be questioned that a crevasse affords an immediate relief when the river is surcharged and the containing levees are water-soaked or insufficient in elevation or stability; but every crevasse also results in a depletion of the capacity of the main channel, which needs to be enlarged rather than reduced.

If the problem were merely to deal with the control of floods for a few years, perhaps spillways or outlets would serve; but the problem is to provide for all posterity, and it appears to the writer that the advantages which may be gained by spillways are not only temporary, but that they are at so enormous a cost for the future that it would be an engineering mistake to formulate designs of flood control based upon a system of spillways.

For the reasons stated, the writer is of the earnest conviction that levees, with auxiliary works for the protection of banks from caving, will afford the only certain method of controlling the floods of the Mississippi River.

IMPROVEMENT OF NAVIGATION IN RELATION TO FLOOD CONTROL

By Stuart C. Godfrey,* M. Am. Soc. C. E.

I.—Navigation and Flood Control

"And the waters prevailed, and increased greatly upon the earth; and the ark went upon the face of the waters." This passage from Genesis is the earliest record that the writer finds of the relationship of floods and navigation. In the modern parallel that occurred in April, 1927, Noah's one ark had increased to a hundred, freely and instantly made available by the Army, the Navy, the Coast Guard, the Lighthouse Bureau, and the various commercial boat lines. In place of the dove, fortunately, the seaplane and the radio were available. And the outcome proved anew, if proof were needed, that these boats cannot be spared from the rivers.

To-day, on the Lower Mississippi, navigation is overshadowed in importance by flood control. The latter, like a modern prodigal son, after a season of wildness and dissipation, comes back to be the center of popular attention. Navigation is the elder brother who once enjoyed pre-eminence on the river, but who now finds himself relegated, at least for the present, to a subordinate place at the feast. All this is natural enough; a wild animal must be tamed before he can become a useful beast of burden. And the elder brother may accept his position the more gracefully, inasmuch as his brother's salvation will bring to him also great benefits.

II.—Present Extent of Navigation

Indeed, navigation on the Mississippi River does not lack encouraging developments. The present volume of this navigation is not generally appreciated. Consider the giant steamer, *Sprague,* so useful last April in rescue activities, at her usual task, supplying the Standard Oil Refinery, at Baton Rouge, La., with crude oil from the Smackover Fields. On March 21, 1926, the *Sprague* finished a 300-mile tow at Baton Rouge with a cargo of petroleum (224 000 bbl. of crude oil) contained in 19 steel barges. (See Fig. 4.)

This tow, the size of four city blocks, contained more than could be carried in the largest tank steamer; the equivalent of no less than 28 full train loads of 40 cars each, or one solid train 9 miles long. It would have taken about thirty of the largest old-time packet-boats to carry this one cargo. No wonder the rivers do not appear so busy in this era of barges and towboats! Yet the Secretary of War was able to state in his Annual Report for 1926 that the tonnage now handled on the Mississippi River System is twice what it ever was in the "good old days" of river steamboating.

* Maj., Corps of Engrs., U. S. A.; Dist. Engr., Dredging Dist., Mississippi River Comm., Memphis, Tenn.

The single traffic in crude oil referred to, handled by the *Sprague* and half a dozen other towboats, amounted in 1926 to 2 500 000 tons, about the equal of the entire foreign commerce of Boston, Mass., Newport News, Va., or Galveston, Tex.

The tonnage on the Mississippi and Ohio Rivers has increased greatly since 1920, as shown by Table 29.

TABLE 29.—SOME COMMERCIAL STATISTICS.

	ANNUAL TONNAGE.		
	1920.	1923.	1926.
Cairo, Ill., to Memphis, Tenn.	1 044 945	1 048 322	1 660 188
Memphis, Tenn., to Vicksburg, Miss.	925 763	1 452 837	4 792 780
Vicksburg, Miss., to New Orleans, La.	2 874 190	5 493 297	11 074 488
Ohio River	9 869 325	9 245 647	19 700 000*

* Approximately.

The Inland Waterways Corporation, in its Mississippi-Warrior Service (operating on the Lower Mississippi and Warrior Rivers), charging a rate which is 80% of the rail rate, carried 1 341 000 tons of freight in 1926, an increase of 18% over the preceding year, with a direct saving in rates to shippers of more than $2 000 000. (See Fig. 5.) This season (1927) the Service has been extended to the Upper Mississippi, with three new towboats of lighter draft now in commission. Promising experiments in the use of powdered coal now being tried on the towboat, *Illinois,* may lead to revolutionary changes in power plant design for river vessels. This corporation, Government-owned, showed in 1926 for the first time a net profit of $219 512, and is being offered more tonnage than it has equipment to move.

The Jones and Laughlin Company is now enlarging its $350 000 terminal at Memphis, Tenn., where it handles annually, at a great saving, 50 000 tons or more of steel products shipped by river from the Upper Ohio. This Company has not only issued an attractive and illuminating pamphlet, "Our Runaway Rivers—Put Them to Work", but is practicing what it preaches.

The Carnegie Steel Company is one of several concerns that are making an increasing use of the Ohio and Mississippi for the shipment of heavy tows of steel from the Pittsburgh District by water to the Gulf. This through traffic can be expected to increase greatly with the completion of the Ohio River improvement project by 1929. The W. C. Kelly Barge Line has built and is putting into operation a fleet of modern Diesel towboats of great capacity. Even the old packet-boats, in general an outgrown type, have in some cases found a successor in such fine modern passenger boats as the *Cincinnati,* plying between Cincinnati, Ohio, and Louisville, Ky.

As for terminals, there is a growing appreciation of the vital need of adequate terminal facilities, finding expression in the rapid development at numerous sites of such structures as the river and rail terminal at Memphis, representing an investment by the City of more than $2 000 000.

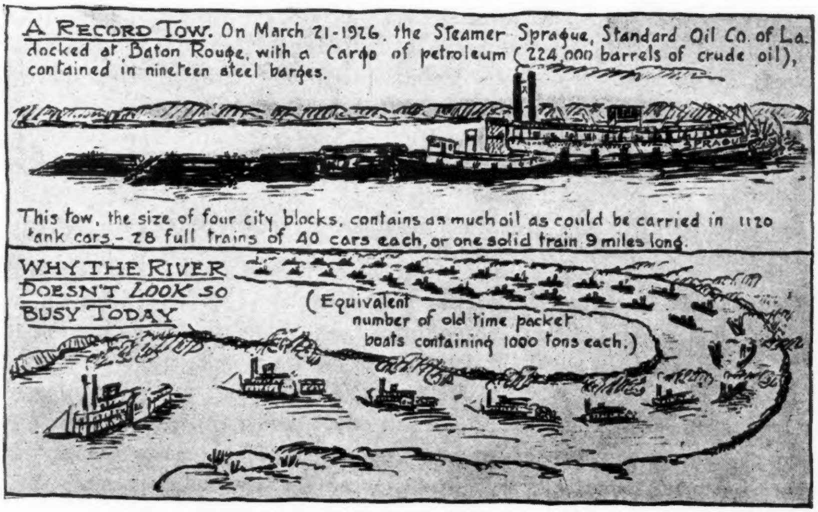

FIG. 4.—VISUAL COMPARISON OF PRESENT AND FORMER TRAFFIC HANDLING.

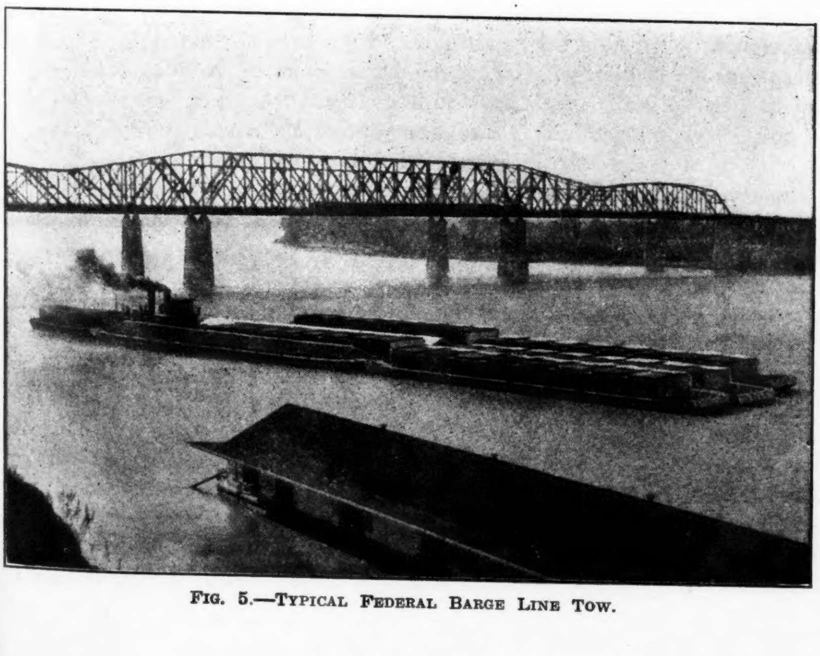

FIG. 5.—TYPICAL FEDERAL BARGE LINE TOW.

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The economics of transportation on these rivers, especially the Ohio, has been discussed* before the Society, and will not be here reviewed. However, the symptoms referred to, relating to navigation at present, are not those of a languishing patient. Rather do they give encouragement to Mr. Hoover's vision of a 9 000-mile consolidated system of inland waterways serving twenty States, developed to keep pace with the vast expansion of transportation facilities that will be required in the next twenty-five years.

III.—Improvement of Navigation on the Lower Mississippi

The scope of this paper precludes more than a brief discussion of some of the practical aspects of maintaining a navigable channel on the Lower Mississippi. The Mississippi from Cairo to the Gulf is about 1 060 miles in length; the lower 290 miles below the mouth of the Red River, has ample depth for river craft at all times.

Among the factors which make the Lower Mississippi difficult for navigation are the instability of its channel, the swiftness of its current in places, its easily eroded banks, and the constantly shifting bars. The bar-forming material is supplied from the eroded banks, which give up annually to the river nearly 1 000 000 000 cu. yd. The most favorable factor, as compared with other rivers, is the large low-water discharge, which seldom falls below 100 000 sec-ft.

The general formation of the river is not essentially different from that of other alluvial streams, with abundant water in the bends, which are separated, however, by long bars stretching obliquely across the river. At the broad "crossings" over many of these bars, the same river that at flood stages measures 60 ft. or more on the gauge, fails by its own efforts to maintain a low-water channel more than 4 or 5 ft. deep. The location of these bars shifts from season to season, and even from day to day. The one unchanging thing about the river is its changeableness.

The first comprehensive plan for improving the Mississippi River was by means of regulation. In 1880, when the Mississippi River Commission made its preliminary report, river navigation was a controlling factor, and the plan of improvement comprised the contraction of the waterway and the protection of caving banks, in order to scour out and maintain a channel through the shoals and to build up new banks and establish a fixed regimen. It was then the general belief, based upon experience with other rivers, that such results might be secured largely through the instrumentality of light flexible structures of poles and brush.

The initial works of river regulation undertaken by the Commission, and those most often referred to, were located in the Plum Point Reach, and the Lake Providence Reach, where low-water navigation was particularly difficult. On these works there was expended over a period of several years between $4 000 000 and $5 000 000, more than two-thirds of which went into contraction works.

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1105.

These works improved navigable conditions, and confirmed the soundness of the general theory of improvement. The frail structures of poles and brush, however, proved too light to withstand long the ravages of the floods. Too little attention relatively was paid at that time to bank protection, and most of the contraction works, lacking continuity and completeness, failed sooner or later through changes in the river resulting in "flanking," if not by direct attack. (See Fig. 6.)

Channel Maintenance by Dredging.—When it became apparent that channel improvement by contraction works, as then conceived, could not meet the pressing needs of commerce within a reasonable time limit, the Commission began experiments with hydraulic dredges of large capacity. After extended study and experiment, such dredges adapted to this peculiar service were developed, and the plan for the temporary improvement of low-water navigation by opening and maintaining channels across the obstructing bars, was applied successfully. This constituted a notable advance in the science of hydraulic dredging. By Act of Congress of June 3, 1896, dredging to obtain and maintain a navigable channel below Cairo of not less than 250 ft. in width and 9 ft. in depth, was made a part of the project.

Since 1895 a navigable channel has been maintained by this method, and except for occasional intervals in periods of unusual low water, the project depth and width have been substantially available. At present, nine dredges are maintained by the Dredging District of the Mississippi River Commission. They have been progressively modernized from time to time, and represent an investment of about $2 000 000. This season (1927) the complete reconstruction of the *Gamma,* now provided with a 1 000-h.p. steam turbine, direct connected to a 32-in. pump, has been accomplished. The cost of channel maintenance below Cairo by this method averages (including all field and repair costs, depreciation, and interest on the investment), about $600 000 annually.

Under the present method of operation, a fleet of survey boats operates as the eyes of the dredging fleet. As the river falls, frequent inspections and surveys develop the more troublesome crossings. To these critical points dredges are despatched in time to anticipate, so far as practicable, the effects of shoaling. Dredging is commenced when depths of perhaps 10 to 12 ft. are still available. It is continued night and day, Sunday and holiday, until the job is finished. The methods of dredging were described in detail in a paper presented to the Society by the late John A. Ockerson, Past-President, Am. Soc. C. E., twenty-nine years ago,* and have not changed essentially.

Other aids to navigation on the Lower Mississippi exist in the form of buoys, lights, and crossing charts. The buoys are placed and shifted as necessary by the survey-boats of the Dredging District, and navigators place much reliance thereon. The lights are maintained by the U. S. Lighthouse Bureau. In 1927, for the first time, navigators are issued charts of all troublesome crossings. These charts show by soundings and simplified contours the shape of the crossing, the direction of the current, and the location of the buoys.

* "Dredges and Dredging in the Mississippi River," *Transactions*, Am. Soc. C. E., Vol. XL (1898), p. 215.

Fig. 7 is a map of the Race Track Chute, 607 miles below Cairo, in the Mississippi River. This chart represents conditions as they existed on September 24, 1927, only. The soundings show actual depths. At mean low water, the Vicksburg gauge is 3½ ft. They are re-issued as often as conditions change, it having been necessary already to issue three successive charts of Corona Crossing.

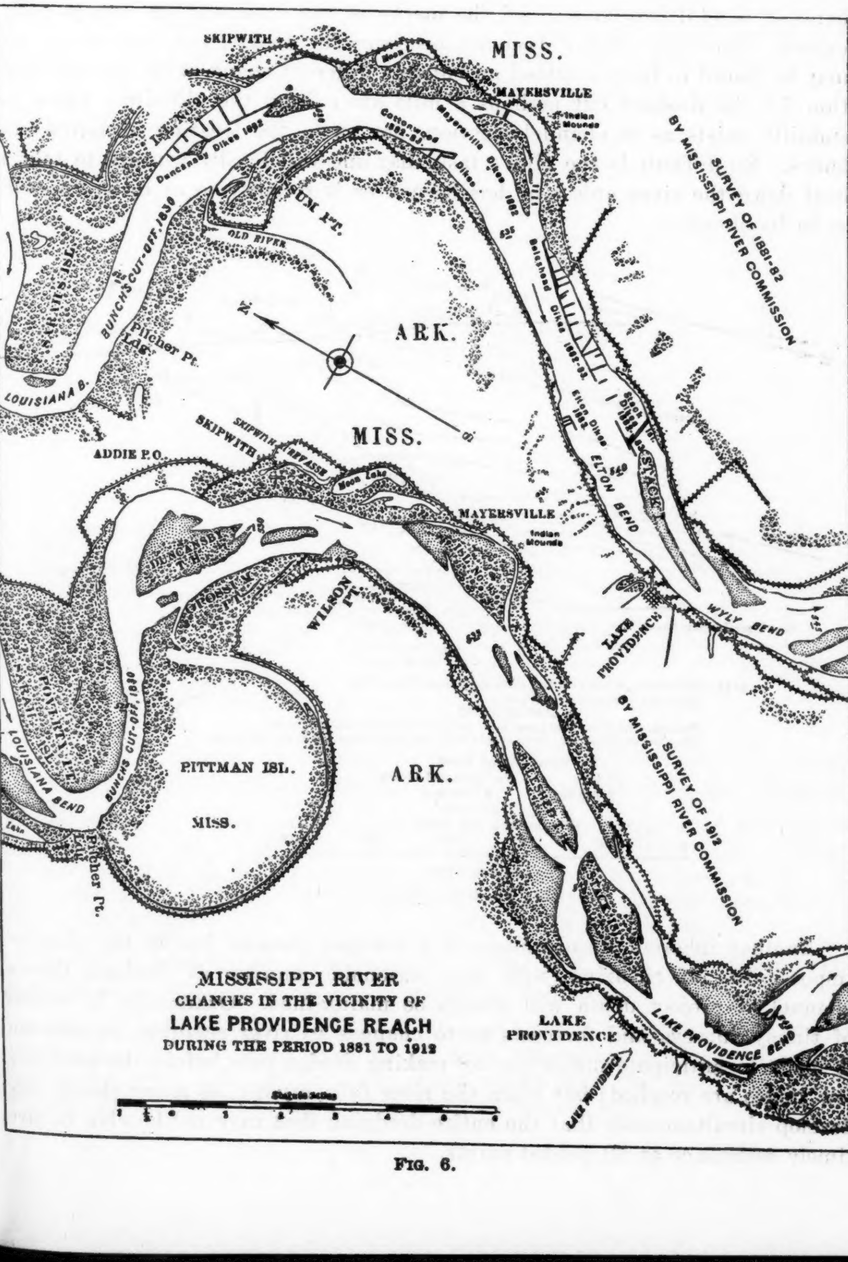

FIG. 6.

Limitations of Dredging.—In this connection, it should be understood that the dredging done for channel maintenance is purely temporary in nature. The excavated material is not removed from the river, but deposited a few hundred feet down stream from the channel limits. If the location of the dredged cut has been skillfully chosen, the channel will maintain itself during the remainder of that particular low-water period. At the first considerable rise of the river, however, the channel fills up much like a path through a bank of wind-driven snow, and the marks of the dredging are largely obliterated. The next time it becomes necessary to dredge, the obstructing bar may be found to have assumed an entirely different shape, with the new location for the dredged cut perhaps a mile away from the old site. Thus, no stability exists as to channel location, nor as to the location of lights and buoys. So difficult is the pilot's task that one will hardly venture to take a boat down the river unless he has been over it recently so as to be "posted" as to its changes.

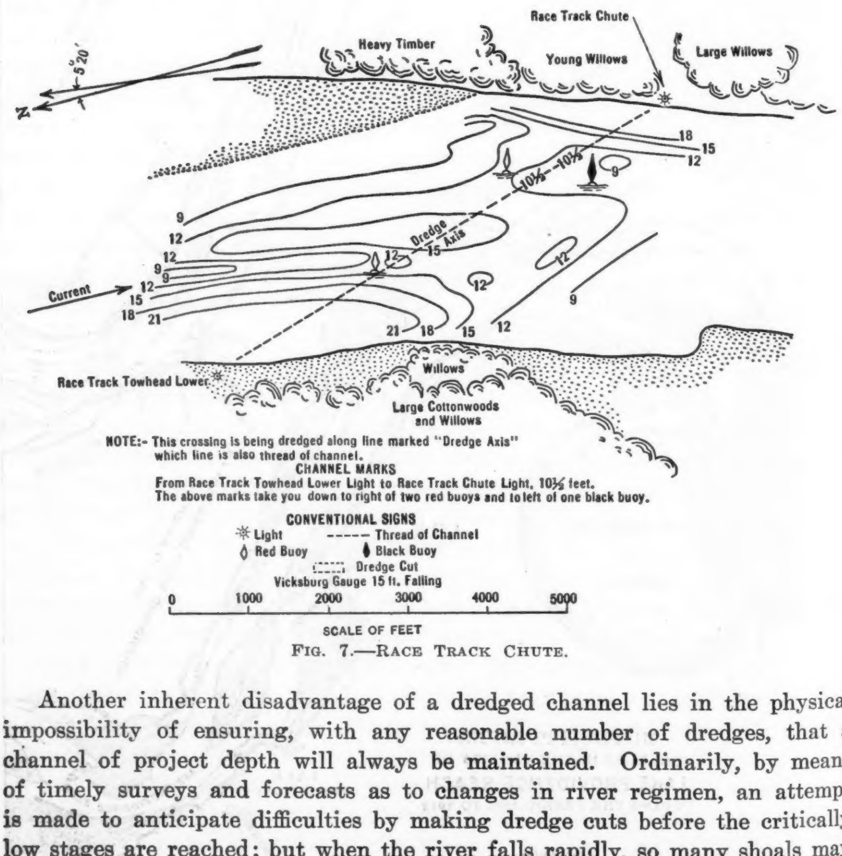

FIG. 7.—RACE TRACK CHUTE.

Another inherent disadvantage of a dredged channel lies in the physical impossibility of ensuring, with any reasonable number of dredges, that a channel of project depth will always be maintained. Ordinarily, by means of timely surveys and forecasts as to changes in river regimen, an attempt is made to anticipate difficulties by making dredge cuts before the critically low stages are reached; but when the river falls rapidly, so many shoals may develop simultaneously that the entire dredging fleet may not be able to give timely assistance at all needed points.

Another serious objection to the present project lies in the restricted width of channel. For the old-time packet-boats a channel width of 250 ft. was perhaps sufficient. The situation has entirely changed with the advent of the present type of towboat and barges. (See Fig. 8.) The normal tow for a Federal Barge Line towboat, for example, is six barges, three abreast, with outside dimensions of 135 by 750 ft. The Standard Oil tows are even larger. Only when such a channel is straight and parallel with the current can it be regarded as affording adequate width. In the occasional sharp bends (see Fig. 9), curved channels (Fig. 10), or those with the current running obliquely to the channel, so as to require "flanking" on the part of navigators, the channel becomes hazardous in the extreme. In such cases, the pilots of large tows usually resort to "double tripping," involving great loss of time.

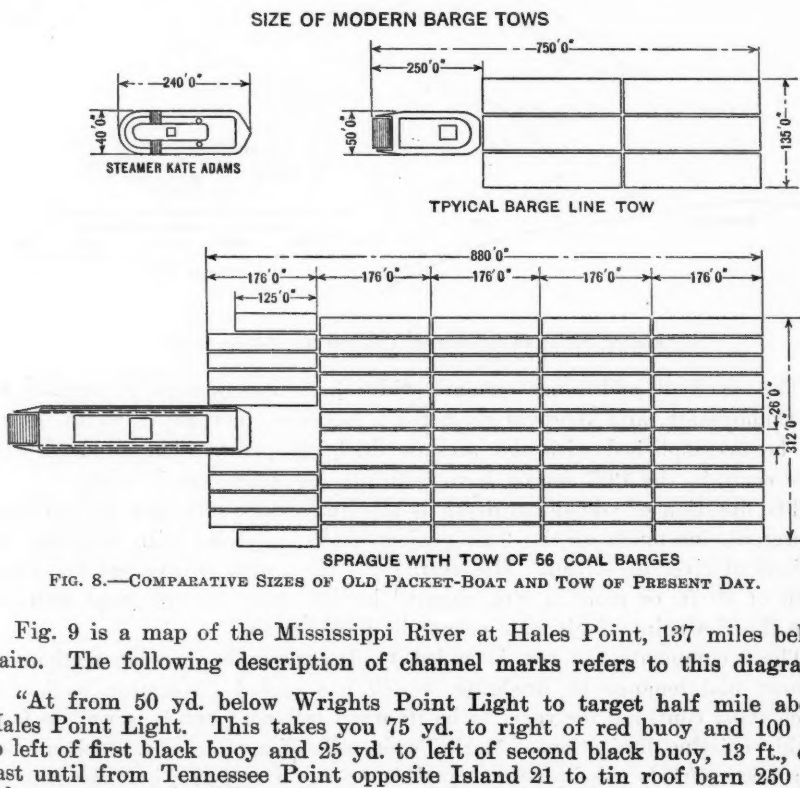

FIG. 8.—COMPARATIVE SIZES OF OLD PACKET-BOAT AND TOW OF PRESENT DAY.

Fig. 9 is a map of the Mississippi River at Hales Point, 137 miles below Cairo. The following description of channel marks refers to this diagram:

"At from 50 yd. below Wrights Point Light to target half mile above Hales Point Light. This takes you 75 yd. to right of red buoy and 100 yd. to left of first black buoy and 25 yd. to left of second black buoy, 13 ft., one cast until from Tennessee Point opposite Island 21 to tin roof barn 250 yd. below target, 15 ft. This takes you to left of third black buoy."

Fig. 10 is a map of the Mississippi at Corona, 204 miles below Cairo. The corresponding channel marks are described as follows:

"From head of white sand below Massey light to 150 yd. below Corona upper light, 13½ ft. Down this way till from 150 yd. below Corona Bar light to Corona lower light, 15 ft. The above marks take you down to right of two red and left of four black buoys, favoring the red buoys."

The soundings were taken July 26, 1927, and July 23, 1927. In each case, the charts represent conditions as they existed on the day of the survey only. The soundings given show actual depths.

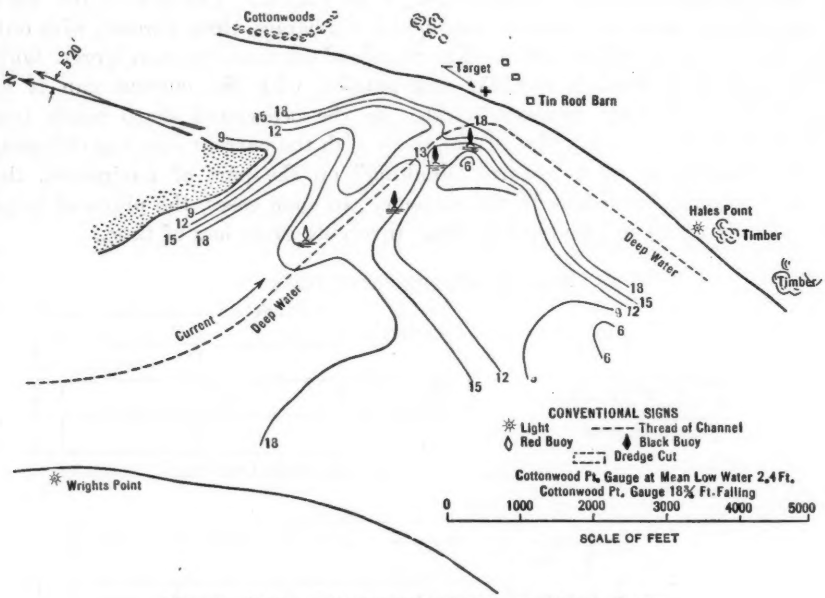

FIG. 9.—MAP OF MISSISSIPPI RIVER AT HALES POINT.

This need of additional channel width has already been recognized by the Commission, and approval given for a moderate increase in width, which can be accomplished with the present dredging plant. The dredged cuts since early in the 1927 season have been not less than 300 ft. wide.

The question of additional depth is also important, although not pressing at present, inasmuch as the 9-ft. project depth coincides with that for the sections of river up stream. However, for a river with an average low-water depth of 30 ft. or more, a 9-ft. channel hardly seems a permanent solution from the viewpoint of ultimate economic development.

These comments are not intended to disparage the present method of channel maintenance by dredging, which has served navigation well, and which must continue for years to be its main reliance; but it is well to face frankly existing limitations. Mark Twain tells of a pilot, one Stephen, who when low of funds was induced to make a trip at a rate just half the regular stipend of $250 per month. The owner's satisfaction at his bargain was shaken when he observed Steve steering the boat up stream against the swiftest part of the current, taking the longer way around the bends, and allowing other boats to pass. His remonstrance was met by the reply that the other boats were handled by $250 pilots, and that he, Steve, knew as much as any man could afford to know for $125. With its dredged channel, the river has doubtless served navigation as well as a $600 000 river knew how;

but after all, such a channel remains at best somewhat of a hand-to-mouth solution, a precarious foundation for a substantial commerce. Some day the growing needs of navigation are likely to demand a better solution.

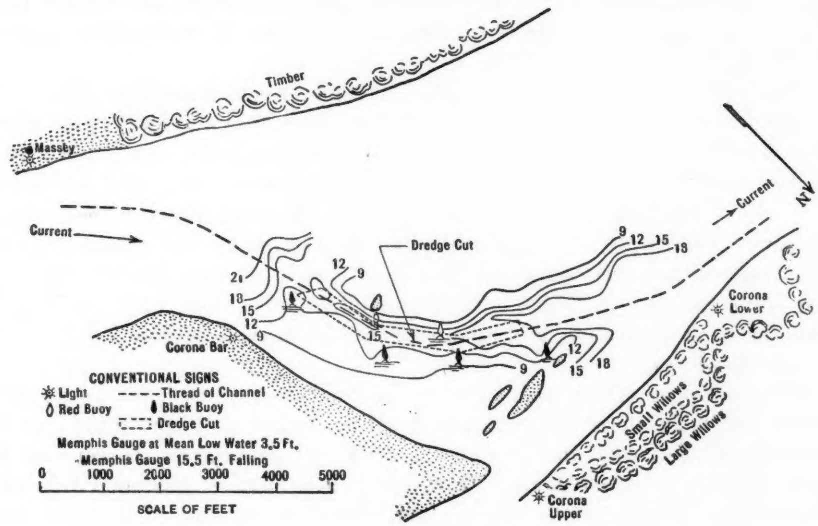

FIG. 10.—MAP OF MISSISSIPPI RIVER AT CORONA.

IV.—AS TO REGULATING THE LOWER MISSISSIPPI

In working toward this more adequate navigable channel, it will be reasonable to draw heavily upon the experience gained in improving the Middle Mississippi (between the mouths of the Missouri and the Ohio). This section of the river has been under improvement by regulating works for many years, the first general project for its improvement having been drawn up in 1872. The present project, as reviewed in 1926, provides for a channel 9 ft. deep and generally 300 ft. wide, widened at the bends, with such supplementary dredging as may be necessary.* As a result of long experience, and after testing many types of structures, the completion of these works is now being carried out on a thoroughly tried basis. The bank-protection works are similar to those developed and used on the lower river. For contraction of the channel, reliance is mainly placed upon timber-pile permeable dikes. Where these dikes are subjected to abnormal river erosion the channelward end is made up of heavy concrete piles. An accretion is rapidly built up by such a dike, resulting substantially in a new bank line. (See Fig. 11.) The resultant contraction of the channel has brought about a marked improvement in navigation conditions.

Studies have indeed been made of such a project for the Lower Mississippi, which is believed to be entirely feasible, given sufficient time and funds. This would involve a progressive treatment over a long period, regarding the

* See H. Doc. No. 9, 69th Cong., 2d Session, "Review of the Project for Improving the Middle Mississippi River."

river as a consecutive whole, shaping its banks and channel with infinite care and giving time for the river to respond. A great advantage of the lower river in this respect lies in its large minimum discharge, and the moderate degree of amelioration required, compared with what might be attained (and which may some day be desired); that is, the degree of contraction required would be comparatively small.

The structures themselves would have to be made somewhat larger and sturdier, on account of the greater dimensions of the lower river. These structures would not be confined to any particular existing types, but would utilize the best types emerging from experimental development work. One promising type which has been used extensively on the Missouri River is the retard, similar to the Brownlow Weed of Indian rivers and the old abattis type on American streams. It consists generally of a series of spur dikes of built-up tree trunks deeply anchored by concrete piles. (See Fig. 12.) It is used both for bank protection and channel contraction, and is now being tested on the Mississippi.

On the Upper Mississippi, with an abundance of coarse sand and gravel readily available along the banks, it has been found that spur-dikes can be constructed effectively and cheaply of sand and gravel, with a capping of brush mattress weighed down with stone. Their success suggested the trial of this type of dike on the lower river, where on account of different soil and conditions it is of far more limited application.

Such a plan of regulation (which would be in fact a return to the Mississippi River Commission's early policy) would provide a channel distinctly superior to that maintained by dredging alone, in width, in depth, in shape, and in permanence. The great cost of complete regulation doubtless renders it out of the question in view of any present need. It is noteworthy, however, that the bank protection required for a comprehensive plan of regulating the Lower Mississippi is estimated to cost no less than four-fifths the total cost; the contraction works necessary for channel improvement comprise the remaining one-fifth. These works of bank protection, moreover, are necessary for flood control and soil conservation and navigation alike; they are, in fact, the basic element in any sound plan of river improvement and control. Thus, bank protection is the great common factor involved in improvements for navigation and flood control. The river must be fixed at its critical points—the caving bends—before any permanent works, including river terminals, are possible. Fix it in whatever shape that is decided upon as best, but pin it down somehow, somewhere. After that, contraction works may be built that will "stay put", if made substantial. In the meantime experiment will have developed the best and cheapest types of structures, both for bank protection and contraction. The willow mattress developed and successfully used for many years in bank revetment, is gradually being superseded by the concrete mat, of which two distinct types have been evolved, both with much promise. (See Fig. 13.) Other types of structures being tested for various purposes of bank protection or channel contraction include the retard and the sand-dike, both mentioned previously.

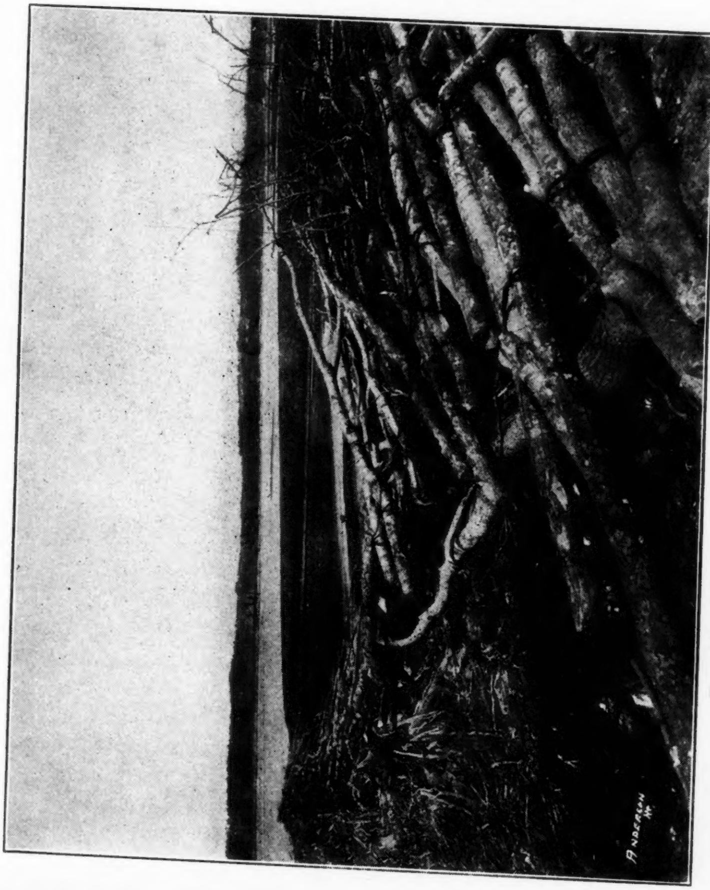

FIG. 12.—RETARD AS USED ON THE MISSOURI RIVER.

FIG. 11.—PERMEABLE DIKE AS USED ON THE MISSISSIPPI RIVER.

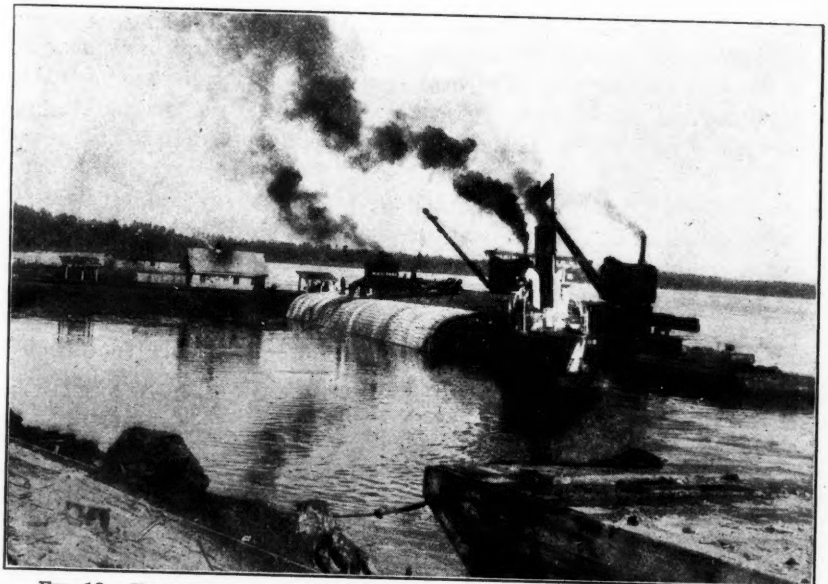

FIG. 13.—VIEW SHOWING THE LAYING OF AN ARTICULATED CONCRETE MATTRESS.

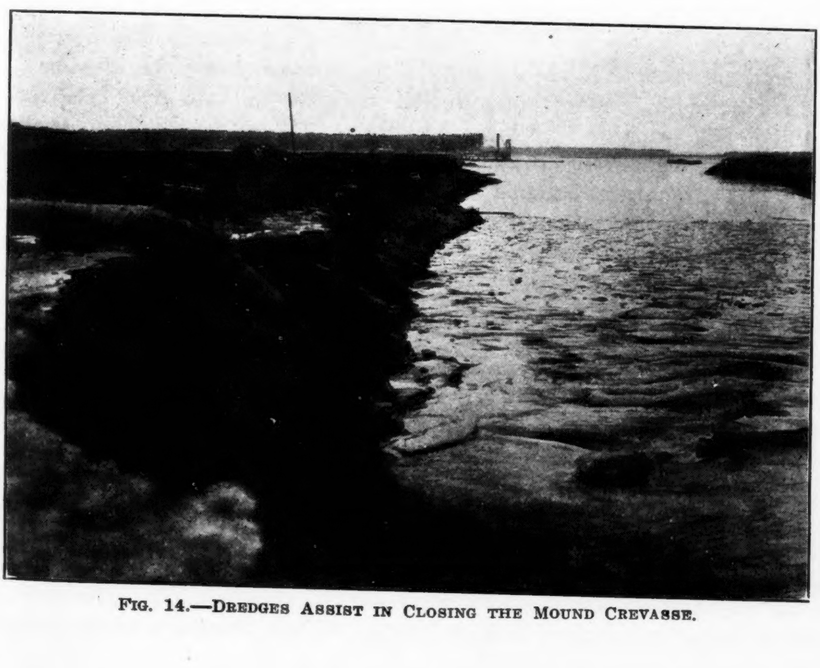

FIG. 14.—DREDGES ASSIST IN CLOSING THE MOUND CREVASSE.

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V.—Relationship of Improvements for Navigation to Flood Control

Navigation and Bank Protection.—As has already been suggested, it can be confidently expected that a complete system of bank protection, whether or not followed by contraction, will materially improve the navigability of the Lower Mississippi. This will be due partly to the exclusion of much of the enormous mass of eroded material from the caving banks, and partly to the greater stabilization of regimen, resulting from fixing the river in the concave bends.

Under the present program of operations, only about 6 miles of bank was protected by new work during the fiscal year of 1927. This is less than 2% of the total amount of bank that will need protection within the next decade. In view of the fundamental importance of this work from every viewpoint, it is reasonable to suppose that with more adequate funds the present program of operations will be largely expanded.

In this important and extensive program of bank stabilization, there will be need for the most careful planning to forward the interests of navigation as well as meet the pressing demands of flood control. In the relatively small amount of bank protection that has been completed, much of it of an emergency nature, it has not always been possible to locate the work so as to serve these various purposes. In laying out a complete system, however, the opportunity surely will not be lost to utilize to the maximum the beneficient effect of bank revetment upon the regimen of the stream. This will mean that some banks will be permitted to wear away before protecting them, that others will be re-shaped to a flatter curvature, that, in short, all work done will be in conformity with, and contributory to, such a plan of complete regulation as may some day be warranted.

Influence of Floods on Navigable Channel.—It is noteworthy that the worst flood-years are also the most difficult years for navigation. Thus, the most troublesome seasons for the last ten years have been in 1922 and 1927, both flood-years. The obvious reason is that the abnormally high stages result in building up the bars to an unusual height (the rate of bar-building being often half as great as the rise of the river itself). When the river eventually falls, and especially if that fall be rapid, the river does not immediately "cut out" a new channel through the bars, but leaves them at higher than ordinary levels. In 1927 it was necessary to commence dredging operations at some crossings when the river stages were still 15 to 20 ft. above mean low water, or about 5 ft. higher than for the average year.

Moreover, the occurrence of crevasses naturally tends to cause deterioration in the channel below the crevasse. Instances of this are numerous, the most obvious evidence of this kind noted in 1927 being at James Bayou Crossing, about 10 miles below the Dorena crevasse. Whereas no dredging has been required over this entire reach for many years, already this season it has been necessary to dredge twice at this point. The more complete control and alleviation of floods would diminish such navigation difficulties.

Influence of Regulating Works upon Flood Heights.—It might be advanced, as an argument against regulation, that contraction works, being in

one sense an obstruction in the bed of the river, would have the effect of raising flood heights. It is well established, however, that they have no such detrimental effect. Such structures are usually low, and the diminution in cross-sectional area which they cause is offset by the increased scour in the channel.

At the International Congress of Navigation in 1900, the consensus of opinion was that flood heights had been reduced upon such rivers as had been fully regulated. The matter is naturally difficult of absolute proof, but this accords with the reasonable hypothesis that with the caving banks completely protected, the rising river finds less material with which to build up its bars, and, therefore, maintains a better high-water channel, tending to draw off the floods more rapidly and to reduce flood heights.

Navigation and Spillways.—It is doubtless true that uncontrolled outlets or diversion channels would be very harmful in their effect upon the navigable channel, comparable to that of crevasses just mentioned. From this viewpoint the general theory as to the advantages of contracting the flow in a single channel is unassailable; but it is significant that the spillways or diversions proposed for the Lower Mississippi contemplate long overflow weirs, with their sills fixed at a level which will not affect low or medium stages, so that the spillways will operate only when relief is needed from excess flood waters. With such spillways, navigation, it seems, can have no serious quarrel.

Navigation and Reservoirs.—The potential capacity of reservoirs to serve navigation is well known, but the practical limitations as to cost and as to the difficulty of utilizing reservoirs for more than one purpose are also generally appreciated. Each case must stand on its own merits.

The studies as to possible reservoir effect now under way are being made primarily from the viewpoint of flood control, and navigation could probably expect only such incidental benefits therefrom as would be in the nature of a by-product. The writer has no data relative thereto, in addition to what have been given, except the statement that if all the reservoirs above Cairo thus far studied were built, they would increase the low-water flow below that point by an amount estimated at 150 000 sec-ft. for a period of 6 months, which would about double the low-water flow, and greatly diminish the dredging required. The great cost of such a system of reservoirs, however, would probably exceed any economic benefit to be derived therefrom.

Navigation in Relation to River Shortening.—In reviewing the report of the Special Committee of the Society on Floods and Flood Prevention,* the writer has been interested in noting the remarks by Messrs. Chittenden, Grunsky, and Townsend, as to shortening the Mississippi. The late General Chittenden thought that the river below Cairo might advantageously be shortened from 125 to 150 miles, or about one-eighth of its length; that the real difficulty in shortening lay in the damage to riparian interests rather than in the disturbance to the regimen of the river. More radical programs of shortening and straightening have also been proposed, some of which go so far as to visualize a mammoth straight ditch with concrete sides. The effect

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1218.

of "improving" in the sense of thus straightening a stream for drainage purposes, was also discussed in a paper presented by L. K. Sherman, M. Am. Soc. C. E., to the Western Society of Engineers, in 1923.*

With the effect of such a program of shortening on flood control this paper is not concerned. However, among the weighty objections advanced against shortening is that of interference with navigation. It appears to the writer that this danger has been exaggerated.

Thirty miles above Memphis is situated Island 35, which divides the river almost equally in two. Of late years the main current, which has long gone the more devious way around the bend, has been tending more and more to take the route through the chute, shorter by 4 or 5 miles. Early in the season of 1927, the Dredging District dredged as hitherto in the bend, with little success as to securing a channel which would remain open. Later, when that channel was plainly playing out, a dredged cut was made in the chute, supplemented by snagging operations. With some initial difficulty this channel has been maintained, and the boats are running there now with increased facility. The criticism that the river had been shortened by causing a cut-off was of course unjustified. The river itself had gradually selected and improved the chute way, and the dredging was merely a recognition at the proper time of that fact.

In this case navigation readily adapted itself to the mildly shortened channel. The pilots of the big river tows, in fact, usually prefer (where there is any choice) the shorter and straighter way; nor do the straighter reaches, on the whole, give greater maintenance difficulty than the crooked ones. A similar case in point is that of Racetrack Chute and Reid Bedford Bend, south of Vicksburg, Miss. The preferable channel, from the navigation viewpoint, is *via* Racetrack, the straight path through the chute, which is a mile or two shorter than around Reid Bedford Bend. It is worthy of note that the swiftest current in this vicinity is found in the point of the bend where the channel is quite narrow. It is not the average current that hinders navigation, but the swift current at some particular spot, as in climbing a reef or negotiating a sharp turn. The average slope below Cairo is less than two-thirds that for the St. Louis-Cairo Section, and both slope and average velocity might be somewhat increased without detriment to navigation, in the writer's opinion, if attention were paid to the critical points.

Suppose, for example, that by encouraging every tendency of the main channel of the river to take the shorter way, as in the two cases mentioned, it proved feasible to gain as much as 1 mile in every 20 miles, or a total of 50 miles. Of course, other critical points would have to be strongly held to prevent a corresponding lengthening of the channel elsewhere; this should be feasible with an expanded program of bank revetment, with which program the shortening would go hand in hand. Such a moderate program of shortening would have better chance of success than a radical one, and would not require the same sweeping and difficult adjustments. Naturally, at flood stages

* "Experiments on the Effect of Upper Channel Improvements upon the Downstream Flood Heights," by L. K. Sherman, *Journal*, Western Soc. of Engrs., Vol. 28, 1923, p. 293.

the river would continue to use both channels, but a greater proportion of the discharge would go the shorter way.

The writer's point is not to contend that shortening is advantageous, but to suggest that the treatment of the navigable channel, which in general departs from the natural regimen as little as practicable, would probably adapt itself without detriment and perhaps with benefit to whatever moderate program of shortening might prove beneficial from a flood-control and drainage viewpoint.

Dredging and Flood Control.—In addition to their normal tasks connected with the navigable channel, the dredges of the Mississippi River Commission have served more directly the cause of flood control. During the past several years the cutter-head dredges have been occasionally useful in filling landside pits and "blue-holes" (thus making levees more secure) and in constructing levees of sand, naturally on a somewhat flatter slope than the standard cross-section. The task of closing the great Mound crevasse is about one-half a dredging proposition, and the dredges there have now completed three-fourths of their task ahead of the original schedule and at a cost greatly below the bids received in response to advertising. (See Fig. 14.) Thus, for any program of levee enlargement or reconstruction, the plant designed to benefit navigation has proved its ability to serve as well the needs of flood control.

VI.—Conclusion

Improvement of navigation and flood control are quite different problems. The former is chiefly concerned with the stream at its lowest stages. The latter has to do with flood discharges thirty to forty times the minimum discharge, and a materially different channel. Flood control seeks to prevent the river from being a destructive enemy; navigation wants it improved in order to make it a useful friend.

Yet, despite these differences, navigation and flood control have much in common, and few if any conflicting interests. A comprehensive plan of bank stabilization is urgently needed for flood control and will greatly benefit the navigability of the river. In this and other ways, by finding a solution to this vast flood-control problem, it will be feasible at the same time to advance the cause of navigation on our largest inland waterway.

THE FLOOD PROBLEM OF NEW ORLEANS, LOUISIANA

By Marcel Garsaud,* M. Am. Soc. C. E.

The flood problem of New Orleans, La., and of the Mississippi Valley below the mouth of Red River, is unique because of the special treatment to which it is susceptible. In order to have a comprehensive understanding of the situation one should be familiar with the topography of the city, and should have an intimate knowledge of the present status of New Orleans as a city, and of its commercial importance as a seaport.

Topography of New Orleans

The City of New Orleans is co-extensive with Orleans Parish, having an area of 196.2 sq. miles, of which 19.7 sq. miles are on the right bank of the Mississippi River. The densely populated and highly developed portion of the city has an area of about 40 sq. miles, and lies in a great crescent-shaped bend in the river. Lake Pontchartrain lies north of the city, the distance from lake to river ranging from 4½ to 7 miles. Lake Borgne, an arm of the Gulf of Mexico, lies to the east of the city. Fig. 15 shows the relation of New Orleans to the water about it. Several canals extend into the city from Lake Pontchartrain. The Inner-Harbor Navigation Canal, with a lock near the river end, connects the river with Lake Pontchartrain.

Except for a narrow strip of high ground along the river bank the ground surface in New Orleans varies in elevation from about 2 ft. below to 8 ft. above Gulf level. Fig. 16 shows the topography of the principal part of the city. (The contours are referred to Cairo Datum. The mean Gulf level is equal to 20.43 ft., Cairo Datum.)

The river at the Carrollton, La., gauge, located at the upper limit of New Orleans, fluctuates from slightly below, to about 21 ft. above, Gulf level. The range of fluctuation would have been increased by the 1927 flood had not the levee been breached at Caernarvon, a few miles below the city. The normal elevation of Lake Pontchartrain is about 0.3 ft. above mean Gulf level, but during windstorms it sometimes rises to a height 5 or 6 ft. above that level. It is therefore necessary that the city be protected against flooding from both the river and the lake.

Protection against the tides in Lake Pontchartrain presents no unusual difficulty. During the storm of 1915 the lake attained a record height of 6 ft. above Gulf level. Levees of very liberal section, with top elevations of 10 ft. above Gulf level, should provide ample protection against flooding from the lake.

The protection of the City of New Orleans against floods of the Mississippi River, however, presents an entirely different problem, toward the solution of which years of study and effort have been devoted and large sums of money have been spent. The river at New Orleans has a width of from 1 800 to

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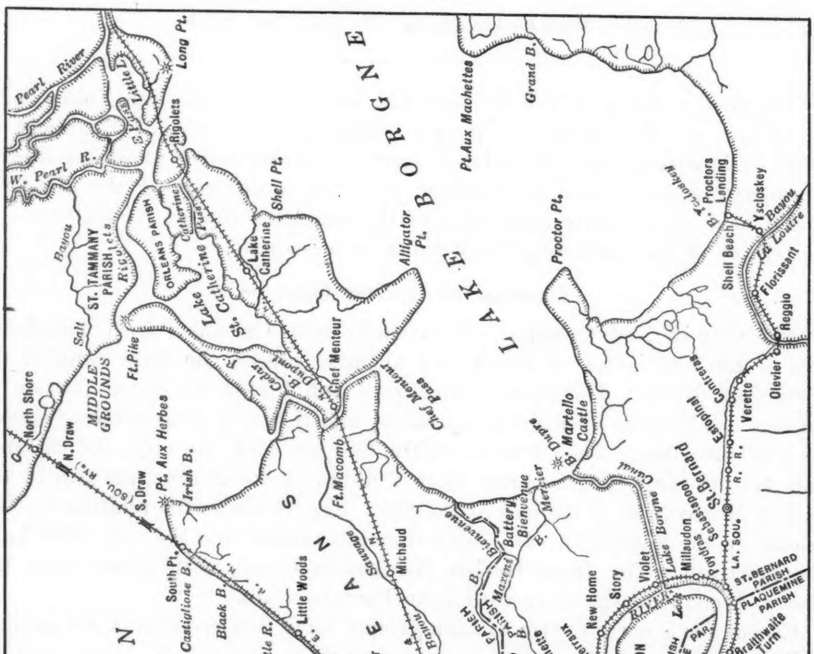

FIG. 15.—MAP OF NEW ORLEANS, LA., AND VICINITY.

3 400 ft. and a depth at high water of from 86 to 194 ft. The maximum discharge measured at Carrollton prior to the flood of 1927 was 1 358 000 sec-ft. and occurred during 1922. The velocity of the current at the minimum section for that discharge was about 7.8 ft. per sec.

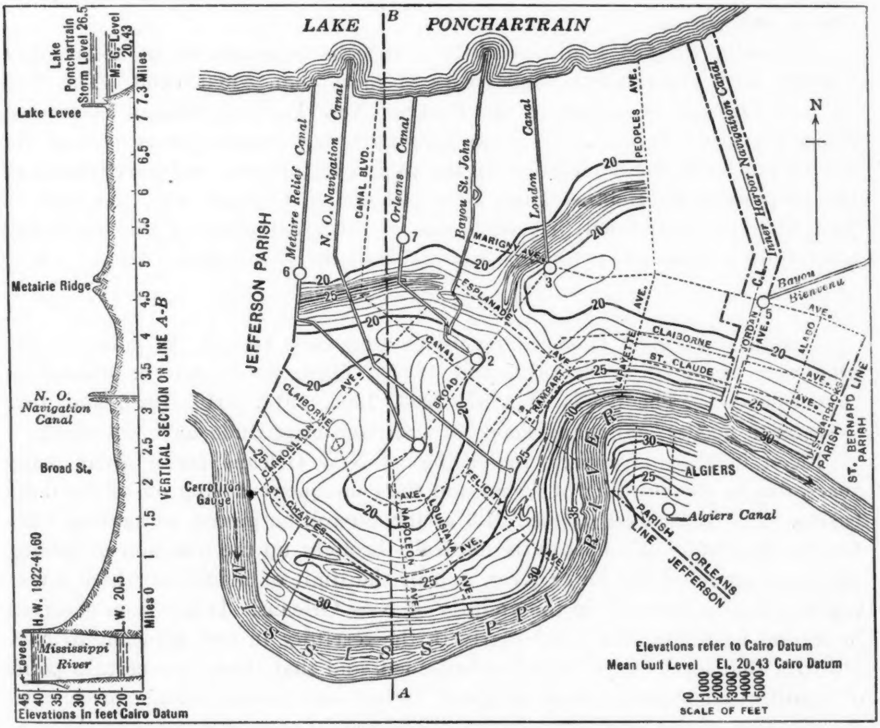

FIG. 16.—CONTOUR MAP OF NEW ORLEANS, LA.

Up to 1924 the protection provided for the City of New Orleans against floods consisted of levees built to a grade established by the Mississippi River Commission, and of such section as would not only be adequate for flood protection, but which would have a land-side slope sufficiently flat to permit of easy access to traffic serving the wharves. In 1924 the Point à la Hache Relief Outlet, which will be described later, was authorized.

The levee grade established by the Mississippi River Commission in 1914 was 25.2 on the Carrollton gauge, or about 4 ft. above the highest water of record. The section adopted for the City of New Orleans and followed, except in particular instances where expediency requires modifications, has a crown width of 50 ft., river-side slope of 1 on 3, and a land-side slope of 1 on 10.

Port Development

The river banks of the City of New Orleans have been given over to commerce and are lined with modern, up-to-date wharves, docks, sheds, and warehouses. On the east bank of the river, out of 12 miles of river bank, 7½ miles

are now covered over by these structures. A great majority of these structures have been erected to conform to the Mississippi River Commission grade as established in 1914, and the State Agency known as the Dock Board, which is charged with the construction and operation of the port facilities, is undertaking the reconstruction of its obsolete wharves to Commission grade as its budget will permit.

The value of port facilities of New Orleans is estimated at about $100 000 000, with outstanding bond issues of more than $43 000 000. The Port of New Orleans is second to the Port of New York in foreign commerce. Steamship lines have sailings from New Orleans to almost every port of the world; and with the completion of the inland waterways, and particularly of the proposed 9-ft. channel from New Orleans to Chicago, Ill., the Port of New Orleans is destined to become one of the great ports of the world and the terminus of all the commerce and traffic of the Mississippi Valley.

Record Floods

The three greatest floods of record passing New Orleans in point of volume were those of 1882, 1912, and 1927. These floods can be classed as "super-floods". The floods of 1897, 1903, 1908, 1913, 1916, 1920, and 1922 were large floods; and many floods of ordinary magnitude have occurred.

Flood heights have been increasing at New Orleans for a great many years, due to the building of levees and the consequent closing off of the delta basins. The 1922 flood reached 21.3 at the Carrollton gauge, as against 14.95 for the flood of 1882. This quite naturally brought up the question of raising the levee grade on the lower river, and of the limit of height to which levees can be built on the soils in the vicinity of New Orleans. It has been observed in several localities, and particularly in the territory in and adjacent to New Orleans, that levees have settled or subsided, and that there is a certain point of equilibrium beyond which height it is not safe to construct a levee. An examination of the levees that have settled brought some engineers to the conclusion that a grade of 25 ft. at Carrollton is about the limit to which the levee grade can be set in this section if a reasonable margin of safety as to load limit is insisted upon.

Relief Spillways

A study of the 1922 flood indicated that much greater floods might occur. It was clearly seen that higher stages would occur at New Orleans in the future. This gave added impetus to the consideration of spillways as a means of lowering flood heights and thereby limiting the height of levees. The lowering effect of crevasses that had occurred during previous floods was closely studied, and a conclusion was reached by a large number of technical men that spillways in the vicinity of New Orleans would give the desired relief. About that time a committee of New Orleans engineers recommended the construction of a spillway a few miles below the city to discharge into Lake Borgne. Diversion of 250 000 sec-ft. was proposed. Due largely to the difference of opinion then obtaining among engineers regarding the effect of spillways, the recommendation was not carried into effect.

In 1924 the Louisiana Legislature passed an Act authorizing the Orleans Levee Board to construct a relief outlet on the left bank of the river below Point a la Hache, Louisiana. The writer was then Chief Engineer of the Orleans Levee District and was charged with the responsibility of reporting on this project. A favorable recommendation suggesting a justifiable expenditure of about \$1 000 000 was made. It was estimated that by completely removing 11 miles of levee about 60 miles below New Orleans, flood heights would be lowered approximately 1 ft. at the Carrollton gauge. The writer further stated that the discharge through the outlet at the crest of a flood like that of 1922 would exceed 250 000 sec-ft.

The Point a la Hache Spillway was constructed at a cost of \$800 000; and observations taken during the flood of 1927 substantially confirmed the ideas of the writer as expressed in his report to the Orleans Levee Board. The curve in Fig. 17, indicates that the stage at Carrollton was 0.8 ft. lower than it would have been without the Point a la Hache outlet. The writer advocated the construction of this spillway not only to relieve flood heights at New Orleans, but also to provide an opportunity to make observations and to study the effect of spillways.

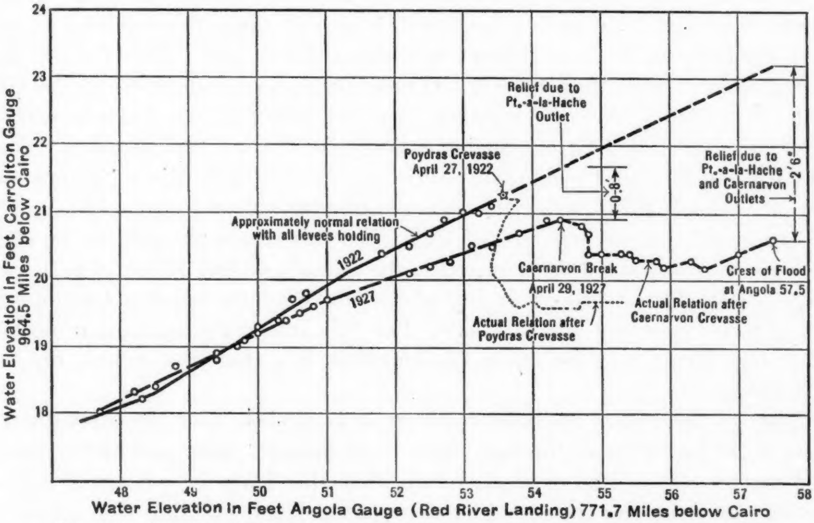

FIG. 17.—ANGOLA-CARROLLTON GAUGE RELATION, FLOODS OF 1922 AND 1927.

The Orleans Levee Board in close co-operation with the Mississippi River Commission made the necessary observations during the flood of 1927. The discharge of the Mississippi River was taken at sections above and below to determine the discharge through the gap. This work was done by the forces of the Mississippi River Commission. The Orleans Levee Board established gauges in the bays at the Gulf end of the outlet and observed them throughout the flood. All these data were furnished to the Federal Spillway Board.

About the middle of April, 1927, when the storms that produced the great flood were occurring, the U. S. Weather Bureau predicted a stage of 24 ft. at

the Carrollton gauge. This was nearly 3 ft. higher than any previous stage. The engineers and business men of New Orleans were confronted with an imminently dangerous condition. It was fully realized that, unless relief should come through a natural crevasse, such as occurred in 1922, emergency action would be necessary. As a result, the left bank levee was breached at Caernarvon about 22 miles down stream from the Carrollton gauge. The discharge through this gap was approximately 270 000 sec-ft. and the lowering effect at Carrollton about $2\frac{1}{2}$ ft. The Caernarvon break not only gave effective protection to the City of New Orleans, but restored faith in the security of the city as well. In addition, it gave another opportunity to make observations and collect more data on mooted points. These observations were also furnished to the Federal Spillway Board.

Permanent Relief Measures

The writer has thus far attempted to sketch briefly the history of the flood question in the lower valley in order that the solution proposed by New Orleans may be more readily understood when presented. The flood problem of the lower valley consists of the necessity for flood-protection works of permanence, adequate not only to protect this part of the valley but also to preserve the integrity of New Orleans as a city and as a port. New Orleans has a population of more than 400 000 and property values of about $1 300 000 000.

It is estimated that the maximum rate of inflow at the latitude of Old River (mouth of Red River) during a flood like that of 1927 would be about 2 500 000 sec-ft. under confined conditions. As the Ohio River System was only in moderate flood at the time it is possible for a flood to occur that would be considerably larger than that of 1927. For this reason the greatest possible inflow is assumed at 3 000 000 sec-ft. at the latitude of Old River. The Lower Tensas Basin forms a reservoir just above Old River in which a large quantity of water is stored during floods. For this reason the maximum outflow below Old River may be taken as somewhat less than the inflow, or, say 2 900 000 sec-ft.

Even to the casual engineer observer it is obvious that the Atchafalaya Basin is the logical place through which to discharge a large part of the excess flood waters. It is suggested, therefore, that this basin be converted into a floodway by removing the levees at its upper end and building side levees to control the flow throughout the full length of the basin. For the maximum assumed flood the Atchafalaya floodway would carry 1 100 000 sec-ft. A controlled spillway of 350 000 sec-ft. capacity would be built on the left bank at Bonnet Carre to discharge through Lake Pontchartrain. This would leave 1 450 000 sec-ft. to pass New Orleans.

Under present conditions, and with levees holding, this flow would produce a stage of not less than 22.5 ft. on the Carrollton gauge. Such a stage would not only endanger the city, but would seriously interfere with the commerce of the port. A third spillway should therefore be built at the Caernarvon site, a few miles below New Orleans. This spillway would be given a capacity

of 250 000 sec-ft. and would lower flood heights at New Orleans about $2\frac{1}{2}$ ft. With such a system of spillways the stage at the Carrollton gauge could be held to 20 ft. under the extreme conditions assumed.

Operation of Proposed Spillways

The Atchafalaya floodway would be uncontrolled and would operate during all floods rising above bank-full stage. In addition to lowering flood heights at and below Old River, it would give a corresponding relief in the Lower Tensas Basin, and would afford a gradually diminishing relief in the main river for a considerable distance up stream.

The other two spillways would be of the controlled type and would be operated only when necessary to prevent unsafe or undesirable conditions. It would be necessary to raise the levees moderately as far down stream as College Point. However, no increase in levee heights would be necessary between College Point and the Passes. The free-board at New Orleans would be 5 ft. for the assumed maximum flood and somewhat more than 5 ft. for such large floods as have occurred in the past.

The spillway at the Caernarvon site will increase velocities along the New Orleans front by reason of the water passing the city at a lower elevation. Too great a diversion at this site might produce scour and caving, and for this reason the capacity of the spillway is limited to 250 000 sec-ft., and the spillway built as a controlled type so that the amount of diversion may be regulated at all times. Velocities of about 8.5 ft. per sec. occurred at the smallest sections of the channel through the city during the flood of 1927. Higher velocities should not be permitted in the future. Recent soundings taken along the harbor front show some scour, which is sufficient indication that greater velocities than those that occurred during the last flood should never be permitted.

In constructing the spillways, auxiliary works of drainage, railway, and highway re-adjustments, and various other works would have to be done, of course, none of which presents unusual difficulties. Fig. 18 shows tentative details of a spillway suitable for both the Caernarvon and Bonnet Carre sites as designed by Roger B. McWhorter, M. Am. Soc. C. E., Special Engineer of the New Orleans Levee District.

Expense Involved

The probable cost of the flood protection project outlined would be about, as follows:

Atchafalaya floodway	$50 000 000
Bonnet Carre spillway	12 000 000
Caernarvon spillway	9 000 000
Levee work between Point Breeze and College Point	35 000 000
Total	$106 000 000

Although the writer has not made estimates of the cost of raising levees on the lower river for a project that does not include spillways, it is believed that an expenditure of more than $100 000 000 would be required for all levee work below Point Breeze, exclusive of that in the City of New Orleans. An estimate was made recently to determine the probable cost of raising the levees and the wharves in the city to conform to the levee grade necessary if spillways are not used. The estimated cost of this work is $40 000 000. The writer thinks it is clear that spillways are not only necessary and desirable, but that they will effect a large saving in cost as well.

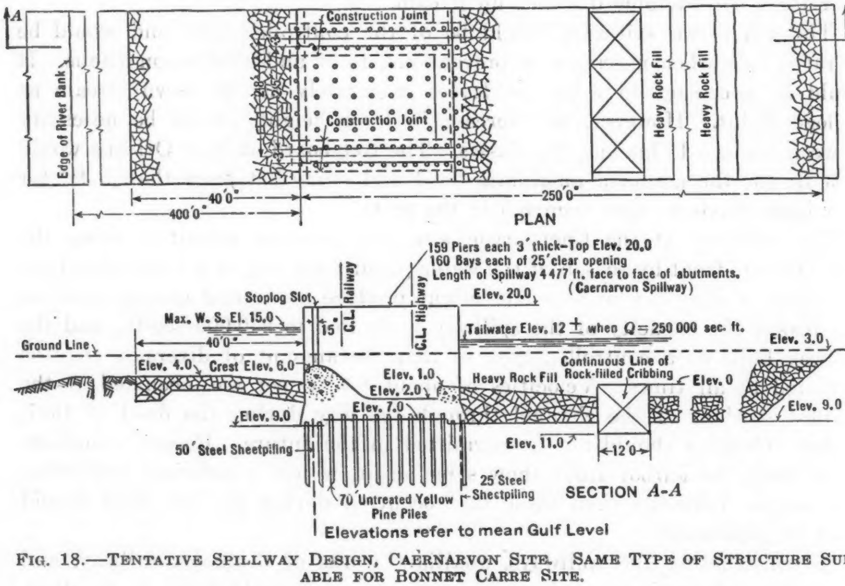

FIG. 18.—TENTATIVE SPILLWAY DESIGN, CAERNARVON SITE. SAME TYPE OF STRUCTURE SUITABLE FOR BONNET CARRE SITE.

The solution that has been given contemplates, of course, that the revetting of the river banks will not only continue but will be carried forward at a more rapid rate in the future.

HISTORIC BASIS FOR SPILLWAYS

The writer has said little as yet in the line of argument for spillways, and little space will be devoted to the subject. To the best of his knowledge and belief there is not an engineer in the country who does not admit that spillways will effectively reduce flood heights. There are a few who believe the main river channel below a spillway will deteriorate as a result of silting. Fig. 17 plainly shows the effect produced by the Point a la Hache outlet and the Caernarvon break on the flood height at Carrollton. The Poydras crevasse of 1922, a mile or so up stream from Caernarvon, produced practically the same effect on the Carrollton gauge as the Caernarvon crevasse of 1927.

Originally, in its natural state, the channel of the Mississippi River was not nearly large enough to carry the flood flow and the excess water was stored temporarily in the delta basins and much of it carried over-bank down

the valley. Then, beginning at New Orleans in 1718, the levee system was built, most of this work having been done during the last fifty years. The delta basins were closed off and the river was confined between levees.

Those who favored "levees only" as a means of flood protection on the Lower Mississippi, as against levees supplemented by spillways, claimed that the confinement of the river would cause the channel to be scoured larger and deeper, and that flood heights would be lowered as a result; but such has not been the case. Cross-sections of the river have been taken from time to time since the Mississippi River Commission was formed, but they show no conclusive change in either direction.

Crevasses have occurred during every large flood, and these diversions have the same effect on the regimen of the river below as a spillway. The writer submits that the channel of the Mississippi River has neither enlarged as a result of confinement between levees, nor silted up as a result of crevasses. Flood flow has little effect in the determination of the regimen of the river. It is some intermediate flow, below bank-full stage, prevailing for a considerable proportion of the time, that determines the regimen. Changes in the channel since man began to take notice of it have been less than the error of measurement of cross-section.

Economic Considerations

There are ample economic reasons for the expenditure of public funds in sufficient amount to construct complete and permanent flood protection works on the Mississippi River. A recent estimate places the 1927 flood damage in Louisiana at \$81 000 000. For each 20-year period in the future one great flood, comparable to the greatest floods of the past, and three other large floods, may be expected to occur. Placing the damage for the one great flood at \$81 000 000, and the damage for the other three floods at \$15 000 000 each, the calculable loss in Louisiana during a 20-year period woud be \$126 000 000.

The loss to Louisiana, due to the fact that investment capital would certainly lose faith in the lower valley, would probably be several times that amount. There can be no doubt of ample justification for the expenditure of more than \$100 000 000 for flood protection on the 300-mile stretch of river below Point Breeze. There is a broader view, however, than that based solely on economic returns. The flood problem of New Orleans and the lower valley involves the preservation of human life, as well as billions of dollars of property values.

The spillways suggested herein, especially the Atchafalaya Basin floodway, will restore to the river some of the characteristics it possessed when it flowed unmolested and unhampered in its natural state, prior to the time of levees which man found necessary to build to reclaim valuable lands.

THE RELIEF OUTLETS AND BY-PASSES OF THE SACRAMENTO VALLEY FLOOD-CONTROL PROJECT

By C. E. Grunsky,* Past-President, Am. Soc. C. E.

Synopsis

The Sacramento River, throughout a large portion of its course from Red Bluff at the upper end of the Sacramento Valley to its outfall into Suisun Bay, flows between banks which are considerably higher than the adjacent valley land. The channel dimensions of the river are in places quite inadequate to carry the extreme floods, as illustrated by one stretch of river, more than 60 miles long, where only about one-eighth of the water at such a flood stage flows in the river. The remainder, under natural conditions, spilled over the bank into the flood basins which paralleled the river. The flood-control project, taking account of these physical characteristics, includes as essential features, not only relief outlets (spillways), but, also, selected areas between levees, so-called by-passes, to which the outgoing water is confined and in which it is carried down the valley to a re-entry into the river at points below which it has larger capacity. The application of such relief outlets and by-passes to the flood control in Sacramento Valley is here described.

Sacramento Valley

The Great Central Valley of California is a depression about 400 miles long, lying between the Coast Range on the west and the Sierra Nevada Mountains on the east. The northerly portion of this valley is known as Sacramento Valley. The northerly apex of the Sacramento Valley is at Red Bluff, at which point the valley begins at once to widen out, attaining and holding a width of about 30 to 40 miles throughout the lower half of its north and south extent. The Sacramento River enters the valley at its northern apex coursing down a little west of its center and discharging into the waters of San Francisco Bay. (See Fig. 19.)

In its meandering course through the valley the river has a total length of about 250 miles. Its fall from Red Bluff to the Bay is about 240 ft. at its low-water stage and a few feet more at its high-water stage, because the rise at flood is about 10 to 15 ft. more at the head of the valley than at points near the Bay.

The Sacramento Valley has a total surface extent of about 4 250 sq. miles (2 720 000 acres) of which the lands, which under natural conditions were subject to flooding and to occasional overflow during freshets, comprise about 1 000 000 acres, or nearly 40 per cent. Three-fourths of the lands which were thus subject to periodical and to occasional submersion by river and

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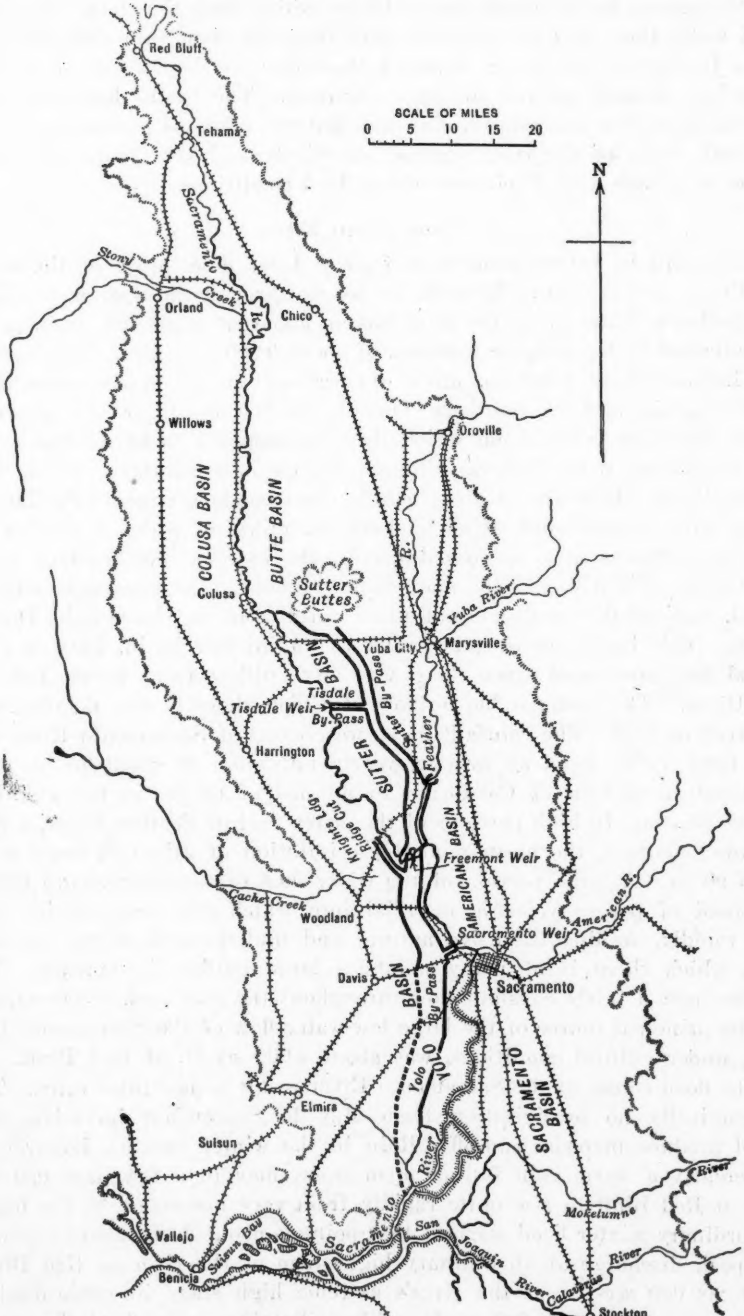

FIG. 19.—FLOOD-CONTROL PROJECT, SACRAMENTO VALLEY, SHOWING SACRAMENTO RIVER RELIEF OUTLETS AND THE BY-PASS SYSTEM.

freshet waters, lie in depressions or basins which flank the river. At times of high water there was an over-bank spill from the river into these basins and on a falling stream water returned therefrom to the stream at a number of points through natural drainage channels. The basins had thus become natural flood-flow retarding basins and had the effect of prolonging the high or flood stages of the river, thereby effectively and very materially holding down the river's peak discharge during flood conditions.

Sacramento River

This will be better understood from a brief description of the original condition of Sacramento River in its course through the Sacramento Valley. At the head of the valley the river has an apparent aggregate drainage area, as indicated by topographic features, of about 9 900 sq. miles. This area does not include about 1 500 sq. miles of territory in the northeastern portion of California and in Southern Oregon, to the northward of the river's visible drainage basin, from which there is supposed to be an underground flow southward, with discharge through springs into tributaries of the Sacramento River. However, it does include the secondary Goose Lake Drainage Basin with a water-shed of about 1 300 sq. miles, of which a portion is in Southern Oregon and the remainder in the extreme northeastern part of California. Pit River, as the main upper branch of the Sacramento River is called, received the occasional output of water from the Goose Lake Drainage Basin. This basin almost takes rank as an interior basin, because a long period has intervened since there was any outflow from Goose Lake into Pit River. The last notable outflow of which there is any definite record occurred in 1868. The rainfall in the water-shed of Sacramento River above Red Bluff varies from an average yearly minimum of about 15 in. on the northeastern plateau of California to upward of 60 in. in the vicinity of Mount Shasta. In high portions of the water-shed of Feather River, a Sierra Nevada tributary, the mean annual precipitation of rain and snow reaches 80 to 90 in. A large portion of the water-shed of the Sacramento River is composed of porous volcanic material into which rain and melting snows sink rapidly, feeding extensive natural and underground storage reservoirs from which there is at many points a large outflow in springs. These springs have a fairly constant flow throughout the year and, in consequence, are the principal source of the large low-water flow of the Sacramento River. This, under natural conditions, was about 4 000 sec-ft. at Red Bluff.

The flood stages of the Sacramento River result from winter rains. There is practically no precipitation from May to September, inclusive, which would produce material run-off. Rain in the winter months, however, and particularly a warm rain falling upon snow, may in a few days cause the river at Red Bluff to rise quite rapidly from very low stages to the highest. The ordinary winter flood stage at that point is about 22 ft. above low water. The peak discharge at an ordinary high stage of the river at Red Bluff is about 200 000 sec-ft.; at the river's extreme high stage its peak discharge may be as great as 270 000 sec-ft. When the river is in flood the extreme discharge at the head of Sacramento Valley does not persist for any great

length of time. The river ordinarily rises rapidly to maximum conditions and then its flow decreases in a comparatively short time to a moderate discharge. Over a period of a week or two the discharge at Red Bluff may be only one-fourth, or even less, of the discharge at the extreme high stage of the river.

From the head of Sacramento Valley to Stony Creek, in a length of 58 miles, the river has an average fall of a little more than 2.5 ft. per mile. In this portion of the Sacramento Valley the slope of the land on both sides of the river is from the hills which encompass the valley toward the river. A number of small streams both from the east and west discharge their waters directly into the main stream. There are here no extensive depressions or flood basins such as are found in the valley farther south. The channel capacity of the river is here sufficient to pass ordinary floods within the natural river banks, and the waters of larger floods which overtop portions of bank lands return to the river channel at points above Stony Creek.

This characteristic of the valley topography does not persist toward the south. The river in its farther course, as far as its delta, has built up high bank lands beyond which are the valley depressions or so-called flood basins. The bottoms of these basins are here and there quite near to, but at other places several miles from, the river. The valley surface has ordinarily a quite uniform gentle slope toward the lowest points of these depressions, giving a smooth appearance to the valley floor.

The stream from Stony Creek to Butte Slough, a distance by river of 53 miles, flows on a lighter grade than in the more northerly portions of Sacramento Valley. Its fall in this stretch is about 1.3 ft. per mile. The river in this stretch cannot carry all the water which comes down in times of flood from the north. Some escapes over bank into the depressions referred to previously. The river's original channel capacity in this stretch decreased gradually with distance down stream from about 150 000 sec-ft. to about 50 000 sec-ft.

From Butte Slough to Feather River, the Sacramento River has a very deficient capacity. Its gradient is here about 0.34 ft. per mile. Under natural conditions the river in this stretch could carry only about 20 000 sec-ft. All the remainder of the flood flow entering the valley from the north, east, and west, above Feather River, found its way into the flood basins and drained from them back into the river lower down. In this stretch the high bank lands are close to the river and there is a rapid fall, in places 15 to 20 ft. into the depressions on either side, making it impracticable to construct river levees far apart.

Below Feather River the Sacramento River in its original condition had a discharge capacity between 75 000 and 100 000 sec-ft., depending on the unknown channel dimensions now modified materially by human agencies, notably by the deposit in the river channel of great quantities of sand which originated in large part in the mountains in the days of hydraulic mining activity. The river gradient from Feather River to the first large partition of water in the river delta, at the head of Steamboat Slough, a distance of 48 miles, is

about 0.30 ft. per mile and somewhat less thence for 18 miles to the point where the water from the great lower west side depression, Yolo Basin, returns to the river and where the capacity of the original river channel increased to upward of 200 000 sec-ft.

Sacramento Valley Flood Basins

For an understanding of the flood-control project on this river it is necessary to say a few words about the Sacramento Valley flood basins which under natural conditions were very effective agents in decreasing the peak of the river's flood discharge. These basins are here very briefly described as they were and as they functioned before the flow of the river had been materially modified by human activities. (See Fig. 18.)

On the east side, in their order down stream, are Butte Basin, Sutter Basin, American Basin, and Sacramento Basin. On the west side are the Colusa Basin (frequently described as consisting of the Upper and the Lower Colusa Basins) and Yolo Basin.

Butte Basin, the uppermost east side depression, extends down stream as far as the contraction between the river's high bank lands on the west and the Sutter Buttes on the east. At this contraction, Butte Slough, a break through river bank lands, forms an interconnection between the river and the basin. The water which flows over the east bank of the river above Butte Slough flowing in a network of channels and uniting with water from a number of streams which drain outlying portions of the Sierra Nevada region, reaches and fills the Butte Basin, the drainage from which, during river flood stages, goes south through the contracted low area between the river and Sutter Buttes already referred to. During a general flood stage of the river this basin holds a slow moving sea of water, 30 to 150 sq. miles in area, depending on the magnitude and origin of the flood waters. The volumetric contents of the basin cannot be given with precision because in times of flood the surface of the water in the basin has more or less slope depending upon the source from which it receives its greatest accession of water.

The second flood basin on the east side of the river is Sutter Basin. This basin has a length from north to south of about 30 miles and an average width of 6 miles. The upper 10 miles thereof, however, has so much surface slope and lies so much higher than the more southerly portion, that, as soon as the inflow of water from the north ceases, there is rapid drainage into the more depressed southerly portion, and submersion of basin lands does not there continue long. The unwatering of the more southerly basin lands being by drainage into Sacramento River near its confluence with the Feather is a slow process. When the flood stage at the mouth of Feather River is at Elevation 30 (above mean sea level), and the basin is full to this height (which is about 8 ft. below the flood-control project high-water stage), its surface has an area of nearly 140 sq. miles and its contents range from 25 000 000 000 to nearly 40 000 000 000 cu. ft., according to the momentary volume of inflow from the north. When, for example, in December, 1889, at the highest stage of the winter, the water in Sutter Basin amounted to

39 000 000 000 cu. ft., or four times the amount which would fill the Sacramento River channel below Iron Canyon from a low-water to a high-water plane. The general elevation of the lowest portion of this basin is 19 to 20 ft. This is but little above the elevation of the low-water plane of Sacramento River at the mouth of Feather River. Complete drainage of Sutter Basin as it was originally, was therefore a slow process.

The river does not fall to its seasonal lowest stage until in the autumn months, and drain-ways to the river which Nature had provided, did not everywhere connect with the lowest spots. Drainage, therefore, was imperfect and water stood in some portions of the flood basin throughout the entire year. It is to be noted that by reason of its position between two rivers, this basin, before its reclamation, received water not only from the Sacramento, but also from the Feather River. While being filled and while discharging its contents, it had a pronounced effect on the flow of the Sacramento River below the Feather.

American Basin is the third east side basin. Its southerly or down stream end is at the American River. Its northerly apex extends along the east side of Feather River for some miles above its mouth. The basin has a surface extent, if measured by such a flood as that of December, 1889, of 110 sq. miles and, at that time, at its highest stage it contained about 25 000 000 000 cu. ft. of water. Its water stage at its southerly end was controlled by the stage of Sacramento River at the mouth of the American. It is interesting to note that a change of 1 ft. in its elevation at a full stage represents as much water as goes by Sacramento City at a flood stage of Sacramento River in 10 hours.

Sacramento Basin is a name sometimes applied to the depressed area on the east side of Sacramento River below Sacramento City. This depression is long and narrow. It has long been so well protected by levees—although there have been occasional floodings—that its having ever had any material effect on the river discharge is generally ignored. There are about 35 sq. miles of the lowest portion of this depression which are only 6 to 7 ft. above mean sea level, and, therefore, about 10 to 15 ft. lower than the original high-water stages in the Sacramento River opposite the upper end of the basin.

Colusa Basin lies on the west side of Sacramento River, being separated from the Yolo Basin by a ridge of relatively high ground built out from the Coast Range to Sacramento River at Knights Landing (Grafton), by Cache Creek. This basin is long and comparatively narrow. Before the water was held back by levees, any general river flood stage converted the entire west side valley trough from a point in the latitude of Princeton to the ridge at Knights Landing into an inland sea nearly 50 miles long and 2 to 7 miles wide. Its water came from the west side over-bank flow of the river and from numerous small creeks which descend from the Coast Range. During flood conditions, when this basin was full, and water was flowing in it from north to south, its contents may at times have reached 45 000 000 000 cu. ft.

Yolo Basin is the largest of the several flood basins along the Sacramento River. It extends from the ridge described at Knights Landing to Cache

Slough at the lower end of Grand Island on the south. It has a length of more than 40 miles and an average width of 7 miles. Its volumetric capacity, before being reduced by reclamation works, was, at times of general inundation when water was flowing down the basin, about 50 000 000 000 cu. ft. Its surface extent may be noted at nearly 300 sq. miles. During the high water of 1889 (this being a flood for which the records permit approximation), the discharge from the lower end of this basin back into Sacramento River through Cache Slough, its outlet channel, was more than twice the quantity of water which the river was carrying. In other words the natural capacity of the river at and below Sacramento was only one-third of the combined flow in the river and in the Yolo Basin and yet this combined flow was not as great as the discharge would have been, at the peak, if there had been no retarding influence by flood basins.

The Flood Control Project*

In view of the situation on the Sacramento River in the Sacramento Valley, as explained, the adopted project was based on the following basic principles:

1.—The river must be forced to carry the maximum amount of the flood flow which can be held between properly placed levees of reasonable height.

2.—The water presented at any time in excess of maximum river capacity must be allowed to escape from the river at selected points under control and only for such length of time as is necessary to keep the river below an adopted flood-plane elevation.

3.—The flooding of lands by the water which is allowed to escape from the river must be restricted to definite areas set apart for the purpose.

In the application of these principles to the Sacramento River, the engineers who advised the State and Federal authorities found it desirable to use river discharges as determined from the flood conditions of March, 1907,* as a general guide and to plan control works with some allowance for safety under similar conditions. Studies made in great detail under the direction of Capt. (now Col.) T. H. Jackson, Corps of Engineers, U. S. A., by the Civilian Assistant, H. H. Wadsworth, M. Am. Soc. C. E., and further amplified by E. A. Bailey, Flood Control Engineer of the California State Reclamation Board, led to an adoption of channel, relief outlet, and by-pass capacities in the main, as follows:

From the head of the valley to a relief outlet weir constructed in 1891, and known as Tisdale Weir, the channel capacity will decrease from about 260 000 sec-ft. to about 72 000 sec-ft. The excess over what the river just above Tisdale Weir can carry goes over the east bank of the river flowing as already described into Butte Basin and thence into Sutter Basin. Tisdale Weir has a fixed crest 1 100 ft. long over which the discharge at flood stage is to be 38 500 sec-ft. (See Fig. 20.)

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 1488–1495.

Below the Tisdale Weir the river is to have a capacity of 33 500 sec-ft. between high levees and the remainder of the flood flow of this part of Sacramento Valley, estimated at 216 500 sec-ft. for a flood such as that of 1907, will be carried in the Sutter By-Pass. To this there will be added the flow of Feather River, 200 000 sec-ft., some miles above the lower or south end of Sutter Basin.

The total flood flow from the north, which will reach the vicinity of the confluence of Feather and Sacramento Rivers, is estimated at 450 000 sec-ft. being the 33 500 sec-ft. in Sacramento River and 416 500 sec-ft. by-pass flow, including Feather River. Of this quantity the Lower Sacramento is expected to take away 107 000 sec-ft., leaving 343 000 sec-ft. to be otherwise cared for. As there is no room for this in the lower river channel, it must be allowed to escape into Yolo Basin. For this purpose a relief outlet controlled by the Fremont Weir has been constructed on the right, or west, bank of Sacramento River a few miles up stream from the confluence of the two rivers. (See Figs. 21 and 22.)

At the City of Sacramento, 20 miles below the mouth of Feather River, the Sacramento receives its last tribuary, the American River. It is estimated that this river at a general flood stage similar to that of 1907, will discharge 128 000 sec-ft. into the Sacramento River. This discharge alone, without regard to the water already in Sacramento River, would overtax the contemplated river capacity (110 000 sec-ft.) below Sacramento. To meet this situation a relief outlet has been constructed near the mouth of American River. It is known as Sacramento Weir. (See Figs. 23 and 24.) For economic reasons this has been placed about 2 miles up stream. Under this arrangement there will be, at times, an up-river current from American River to the relief outlet, but this will be so rare and will continue for such a short time that no bad effects, such as channel obstruction by silt, are feared. It is anticipated that under recurrence of a 1907 flood considerably more than 100 000 sec-ft. of water would go out of the Sacramento through this Sacramento relief outlet.

At each relief outlet there is a permanent structure replacing a section of levee, through or over which the outgoing water flows. There is thus a Tisdale Weir on the left or east side bank of Sacramento River at the upper end of Sutter Basin; a Fremont Weir on the west bank of the river a few miles above the mouth of Feather River; and a Sacramento Weir also on the west bank of Sacramento River 2 miles above its mouth. The crest of the Tisdale Weir is at an elevation about 2 ft. above the general level of bank lands and about 9 ft. below the extreme flood height in Sacramento River. Whenever water reaches the crest height of the weir the outflow from the river begins and continues during flood stages until the water in the river again drops to below the height of the crest of the weir.

The Fremont Weir also has a fixed crest which is at the general level of bank lands. A trench was excavated to receive this structure. The length of its crest is 9 000 ft. Here, as in the case of the Tisdale Weir, flood water flows freely over the weir whenever the flood stage is higher than the crest. When

flood conditions are similar to those of 1907 the depth of water on the weir will be about 7.5 ft.

The Sacramento Weir has a length of 2 000 ft. and is provided with a movable crest. The top of the fixed structure is about 10 ft. below the river flood stage and the movable crest adds 6 ft. to the effective height of the weir.

At all three weirs the force of the over-falling water is broken by a stilling device, a box-like depression constructed of concrete. In the case of the Fremont Weir, on account of the slight elevation of the crest above the land over which the weir discharges, the stilling-box is shallow and a down-stream apron of rock has been added for additional protection.

The water which flows from the river, through the various relief outlets is not allowed to spread out at random over the basin lands. (See Figs. 25 and 26.) It is confined to comparatively narrow channels by levees. The water which reaches Sutter Basin through the Tisdale relief outlet, together with the water that flows out of Butte Basin, is thus forced by levees, 4 000 ft. apart, over toward the east or Feather River side of Sutter Basin (see Fig. 25) and, as already stated, unites with the water of Feather River a few miles above the mouth of Feather River. By means of this arrangement the river's over-bank waters are carried around the main portion of Sutter Basin and about 55 000 acres thereof are afforded protection against the floods of both the Sacramento and the Feather Rivers. The water which flows through the Fremont relief outlet (Fig. 22) and that which flows through the Sacramento relief outlet is likewise confined between levees, making it practical to prevent inundation of a strip of land on the west bank of the river and of considerable areas along the western margin of Yolo Basin.

It would go beyond the purpose of this paper to describe in detail the various areas which have thus received protection against river floods, or to discuss the economic aspects of the Sacramento River flood-control problem. It will suffice to add to the foregoing description that the location of the various by-pass areas necessitated the construction of some quite high levees. Thus, for example, in the case of the Sutter By-Pass there are long stretches of levee from 20 to 25 ft. in height. At times of flood the water of the by-pass stands against these levees for days at a time and subjects them to the usual danger in such circumstances of being attacked by wave action or threatened by seepage waters.

The by-passes have considerable width. The one through the Sutter Basin, as noted, is 4 000 ft. and more in width. That through Yolo Basin ranges from 7 000 to 14 000 ft. When they are filled with water this great width gives the wind a free sweep and subjects the by-pass levees to attack at exposed points by wave action, making their protection with facing of rip-rap or concrete at exposed points necessary.

The fact that high levees are vulnerable is generally recognized. This is particularly true where as in the case of the California by-pass levees there is a long period each year in which there is no rainfall, and in which, therefore, the levees become very dry and subject to cracking where clay enters into their composition. Absolute assurance of protection behind high levees can-

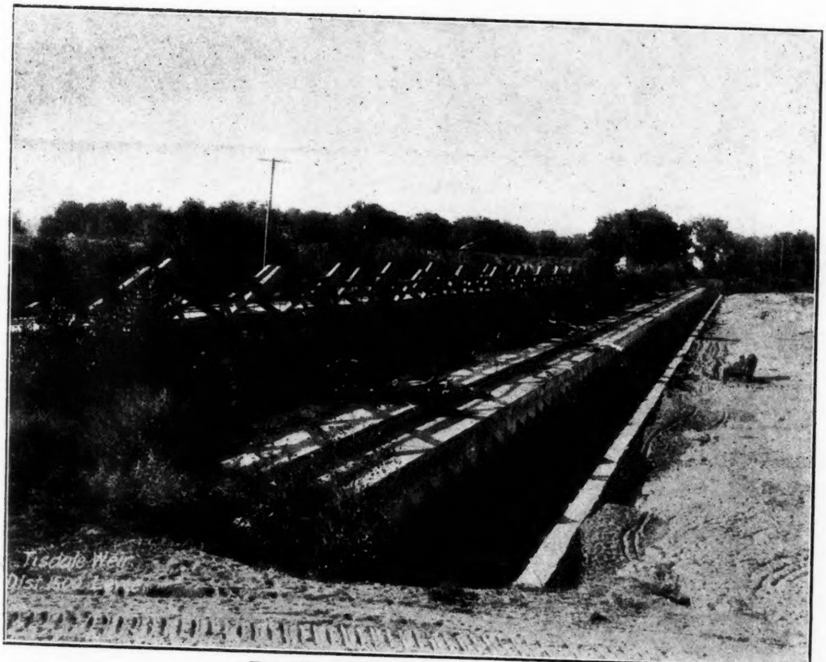

FIG. 20.—VIEW OF TISDALE WEIR.

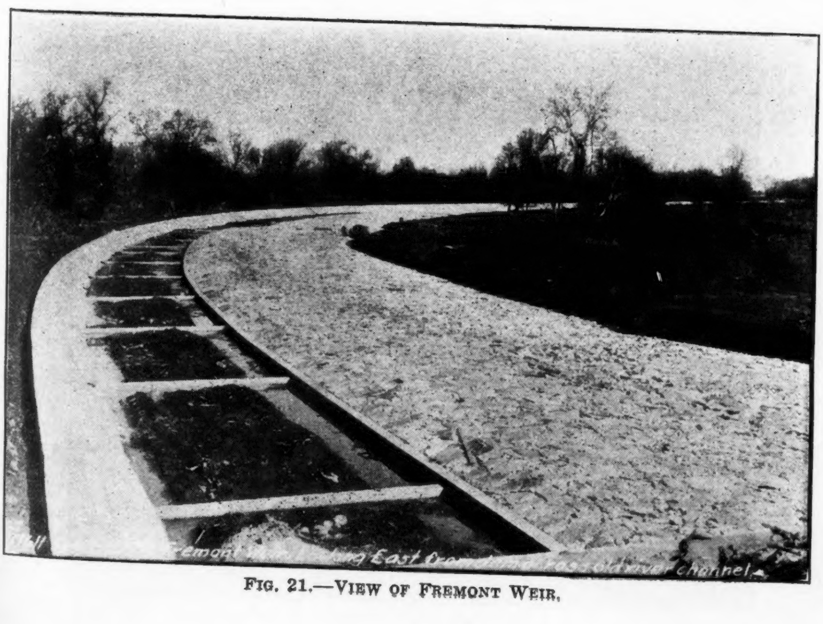

FIG. 21.—VIEW OF FREMONT WEIR.

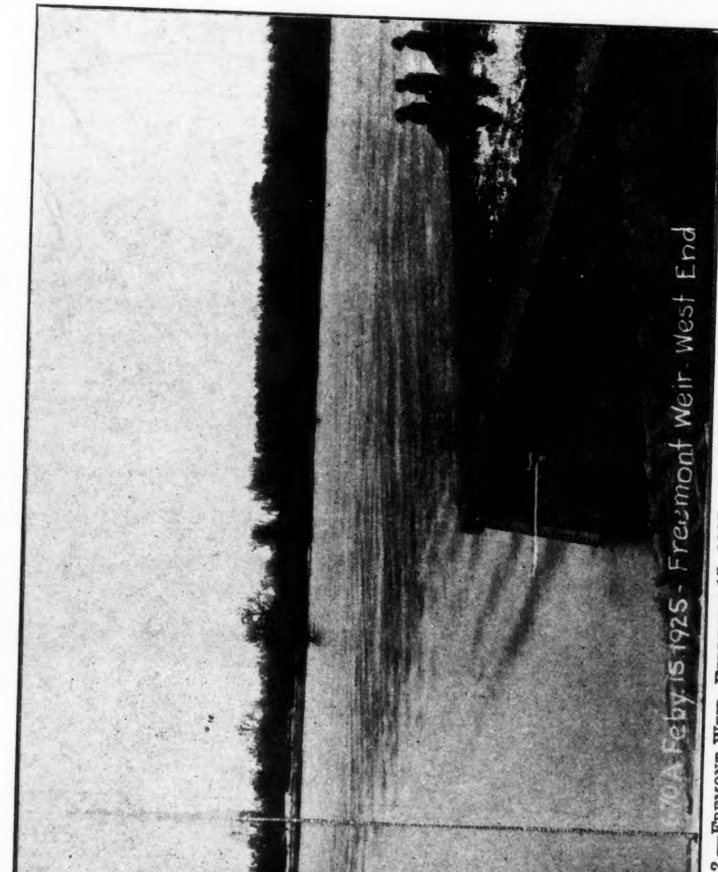

FIG. 22.—FREMONT WEIR, FEBRUARY 15, 1925.

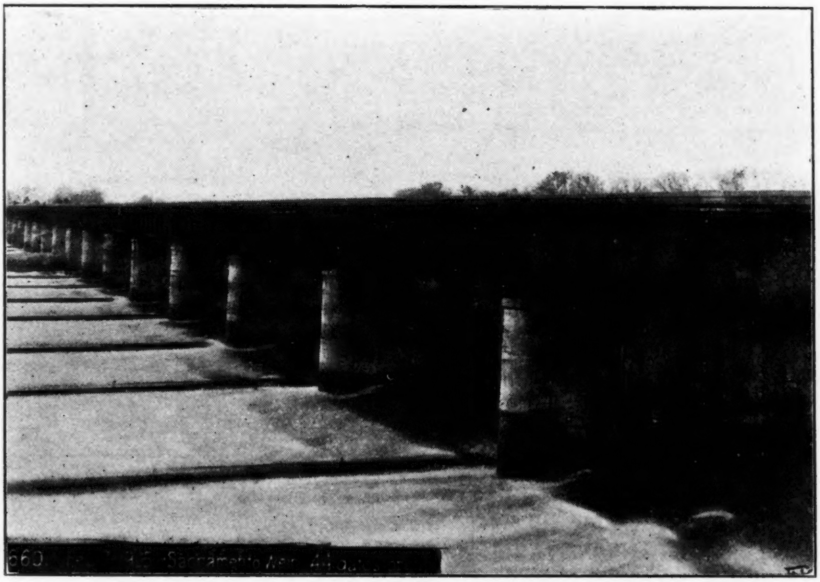

FIG. 23.—VIEW OF SACRAMENTO WEIR, FORTY-FOUR GATES OPEN.

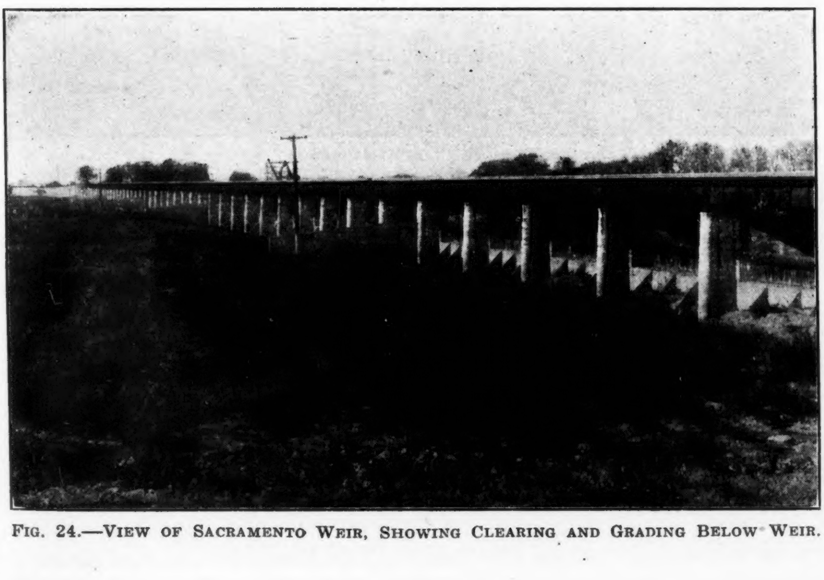

FIG. 24.—VIEW OF SACRAMENTO WEIR, SHOWING CLEARING AND GRADING BELOW WEIR.

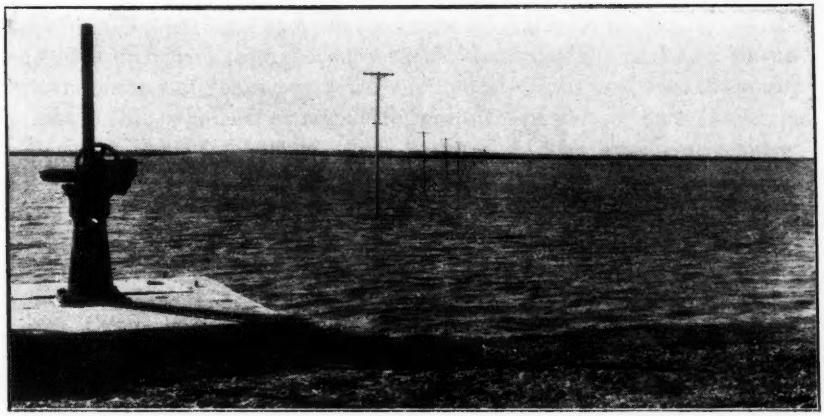

FIG. 25.—SUTTER BY-PASS OPPOSITE TISDALE BY-PASS.

FIG. 26.—VIEW OF SACRAMENTO BY-PASS, FEBRUARY 6, 1925.

FIG. 27.—VIEW OF SACRAMENTO RIVER, SHOWING DOLAN BREAK IN WEST LEVEE, BELOW COLUSA, CALIF., FEBRUARY 4, 1915.

not, therefore, be given. (See Fig. 27.) When this is taken into account, together with the additional fact that volumes of water to be handled at the extreme flood cannot be known, because any flood stage of the past may be surpassed in the matter of volume of flow sometime in the future, then it appears quite questionable whether attempts to protect completely such areas as Sutter Basin should ever be made. This basin might well have been acquired by the State of California and held for all time as an overflow recipient functioning as a retarding basin. Had this been done much of it would have been free of overflow in May or early in June of most years and could have been farmed to summer crops. There would have been little or no economic loss to the State. Furthermore, the flow into this basin and the discharge from it would have prolonged the duration of floods and would have cut down the peak discharge of the lower river, thereby simplifying and rendering safer the protection works on the lower river.

The Sacramento Valley flood-control works have thus far functioned under moderate tests only. There has been no flood stage comparable with that of 1907, nor even with that of 1909, since the completion of the two lower relief outlets. There has, however, been water in the by-passes, notably in February, 1927, when it reached an elevation 10 ft. below the levee tops in the Sutter By-Pass. All that can be said at this time of the relief outlets on the Sacramento and of the by-passes is that their introduction as features of the flood-control works was based on sound principles, but that it is still questionable as to whether these principles have been throughout properly applied to the existing situation.

That the flood-control problem in the Mississippi Valley is comparable with that in the Sacramento Valley seems to be established by recent events. When such volumes of water are presented as was the case during the 1927 floods in the Mississippi Valley, the peak river discharge would demand a waterway far in excess of what can everywhere be provided between river levees of reasonable height. Moreover, no one can give assurance that some time in the future there may not be even a worse combination of discharges from tributaries and still greater demand for capacity. It is apparent, therefore, that the present program of levee building should be approved and accepted as a first step in the execution of a comprehensive project, to be, however, supplemented with relief outlets—always preferably in the nature of spillways with a high crest level—through which the river can unload its surplus water.

At such relief outlets no water should go out except when the river rises above a predetermined danger stage, and the discharge through them should cease as soon as the river is again below the danger line. The outgoing volume of water and the flooding of lands will thus be kept at a minimum as will also the number of days or hours during which the outflow continues. Moreover, the areas that will be flooded should be selected in advance, as in the case of the Sacramento River by-pass areas, and they should be held in public ownership in order that farming activities thereon can be kept under control and erection of improvements that would be too greatly damaged by occasional floods can be prevented. It will readily be seen that under such

a plan great stretches of levee, perhaps miles in length, can be held relatively low and converted into overflow weirs or spillways by being covered with concrete, and yet will serve as effectively as the remainder of the river levee in confining the river at a reasonably high stage to its channel. When an extreme flood flow is then presented, the outgoing water will not be discharged through deep accidental breaches in excessive volume (see Fig. 27), but will go out at selected points over high barriers and will be held at the minimum amount which will afford the desired relief.

Cost of the Project

This project is not merely a flood-control project. The lower end of Sacramento River below Cache Slough is being widened and deepened and navigation conditions there and elsewhere along the river are being protected and improved. The cost of the project falls upon the United States, the State of California, and the land owners, as indicated by the following figures taken from the report of the State Engineer under date of February 10, 1925:

Total Federal appropriations to June 30, 1924		$3 749 900
State Appropriations:		
Co-operative work	$3 500 000	
Lands, cut-off, and channel enlargement	248 600	
For Sutter By-Pass	3 000 000	
Total by State		6 998 600
Land owners' expenditures and obligations		20 497 804
Total appropriations, expenditures, and obligations		$31 246 304
Estimated cost to complete		20 000 000
Estimated total project cost (since 1910)		$51 246 304
Expended by land owners on works useful in the project prior to 1910		$7 272 526
Expended by land owners within the area protected by this project, on back levees, drainage, and other works necessary for land reclamation since 1850 to December 31, 1924		$41 344 445
The cost of levying assessments from time to time, interest, discounts, and other costs of financing, is estimated by the State Engineer at		$7 657 148
The land owners' expenditures for maintenance of flood control and reclamation works to December 31, 1924, are estimated by the State Engineer at		$8 561 643

Infrequency of Extreme Flood Flow

It is now generally recognized that records of precipitation do not cover long enough time periods to show with positiveness any tendency toward permanent change in the amount thereof or in its intensity. Moreover, the combination of circumstances which produce extreme flood conditions are not of frequent occurrence. Abundant evidence can be found to show that greater floods than any of which there is record are still occurring from time to time and that the possibility of such greater floods should not be lost sight of when flood-control works are being planned.

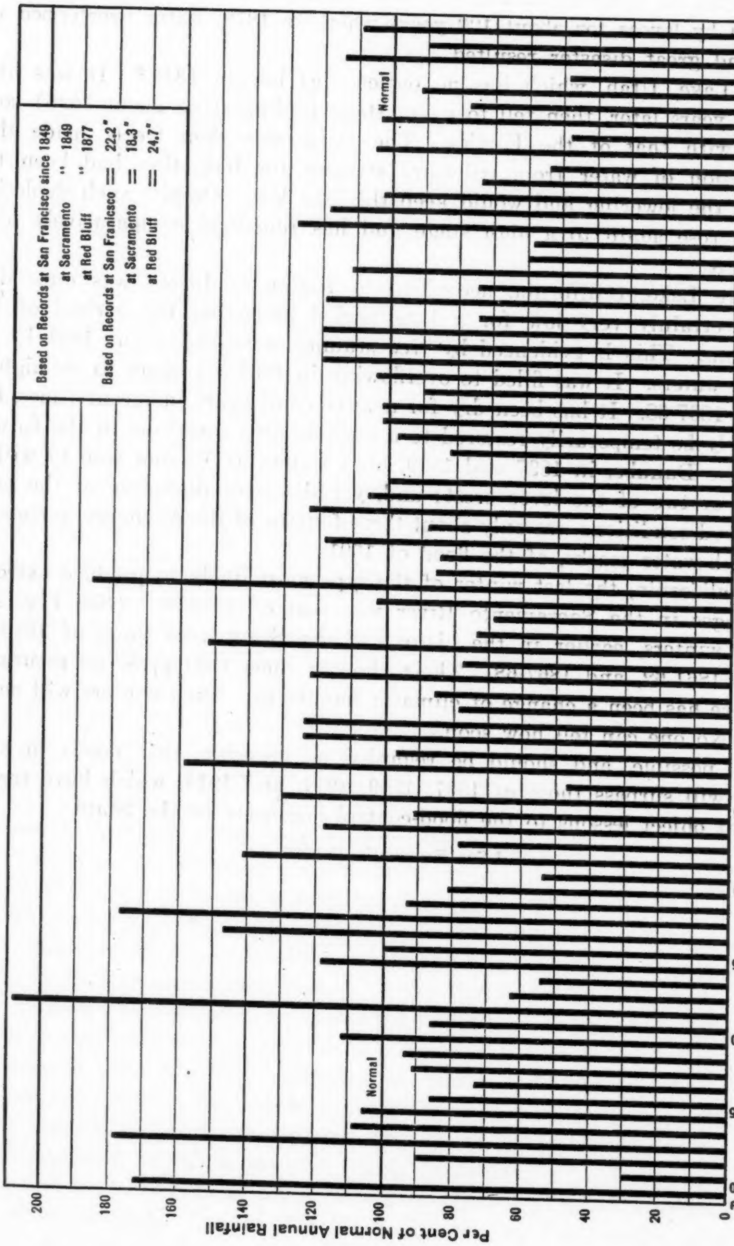

FIG. 28.—RAINFALL IN CENTRAL CALIFORNIA, 1849 TO 1926.

The following facts are cited by way of illustration:

The City of Szegedin, Hungary, on a tributary of the Danube, had been protected by levees for about 100 years when, in 1879, water overtopped the levees and great disaster resulted.

Salt Lake, Utah, which has no outlet, was low in 1843.* It was high 20 to 30 years later, then fell to a low stage (culminating about 1896) comparable with that of the Forties. The claim was then freely made that consumption of water from tributary streams for irrigation had been the cause of the lowering and would keep the lake low. Despite such depletion the lake rose again to a high stage and has remained comparatively high since 1908.

Tulare Lake, California, according to Indian tradition, was once dry. It was certainly very low for a long period preceding the arrival of the white man. This is evidenced by tree stumps uncovered about 1881 by its receding waters. It was filled to overflowing in 1861-62; again to its highest stage in 1867-68. It has been dry for a number of years in recent times, but will surely be temporarily restored to a sizeable area sometime in the future.

On the Danube, in 1897 and 1899, high waters at Vienna rose to within an inch or two of the levee crest; and, yet the river discharge at the peak of the flood conditions was only about three-fourths of the discharge, estimated from high-water marks, of the flood of 1501.

In California, the last winter of the type most likely to produce extreme flood stages in the Sacramento River was that of 1889-90. (See Fig. 28.) Similar winters, earlier in the history of the State were those of 1849-50, 1852-53, 1861-62, and 1867-68. Their absence since 1890 gives no assurance that there has been a change of climatic conditions. Such winters will occur again. No one can tell how soon.

It is possible, and should be regarded as probable, that floods in such winters will surpass those of 1907, 1909, 1911, and 1914, which have served as recent object lessons to the flood-control engineers of the State.

* See Fremont's Reports.

RECLAMATION AS AFFECTING FLOOD CONTROL

By Elwood Mead,* M. Am. Soc. C. E.

According to the 1920 Census figures, there are in the United States a total of 7 538 irrigation reservoirs with an aggregate storage capacity of 21 246 436 acre-ft. Of these, 1601 reservoirs with an aggregate capacity of 6 023 922 acre-ft. are located within the Mississippi River Drainage Basin. All these reservoirs would be filled with about 4½ days' average discharge of 1 375 000 acre-ft. per day, and amounts to less than 2 days' flow of the Mississippi at maximum flood, maximum discharge being estimated at 1 806 000 sec-ft. It is apparent, therefore, that the flood-reducing effect of the irrigation reservoirs now in existence is negligible.

Approaching the problem from another angle, the total irrigated area in the Mississippi Basin (1920 Census figures) is about 5 000 000 acres. Assuming an average diversion of at least 2 acre-ft. per acre, there would be roughly 10 000 000 acre-ft. by which the discharge of the river during the irrigation season has been reduced by reclamation development; and this reduction is effected during the period of the great Mississippi floods, as the return flow arising from the irrigation diversion dam does not reach the stream until after the flood period.

According to the same Census figures, the total irrigable area included in existing irrigation enterprises in the Mississippi Basin is about 10 000 000 acres, and there is, therefore, an apparent opportunity by the extension of these projects to reduce the flow of the river by another 10 000 000 acre-ft. Assuming that this is spread out uniformly over an irrigation season of 180 days, it would effect an average reduction in flow amounting to 28 000 sec-ft.; or about 4% of the average flow of the stream.

On the Mississippi itself, the serious floods arise elsewhere than in the irrigated regions. The effect of reclamation on the control of its floods appears, therefore, to be negligible; its effect on those tributaries lying within the arid regions is, however, a controlling factor. On the North Platte River former destructive floods have been practically eliminated by the construction by the U. S. Bureau of Reclamation of the Pathfinder and Guernsey Reservoirs. These two reservoirs provide 1 140 000 acre-ft. out of the total storage on this stream of 1 800 000 acre-ft. On the South Platte, 338 reservoirs are in operation with a storage capacity of nearly 1 000 000 acre-ft. On the Arkansas, 367 reservoirs provide 1 155 000 acre-ft. of storage, and the effect within the valleys of these streams, as well as other tributaries on which storage has been provided, has been highly beneficial.

Farther west are regions where flood protection by irrigation reservoirs has been most marked. One of the U. S. Bureau of Reclamation reservoirs, namely, the Elephant Butte, on the Rio Grande, was designed with the purpose of setting aside a definite portion of its capacity for flood control. Its total

* Commr. of Reclamation, U. S. Dept. of the Interior, Washington, D. C.

capacity is 2 638 000 acre-ft., of which the upper 400 000 acre-ft. is reserved for this purpose. The rapidly alternating conditions of extreme shortage and devastating flood characteristic of many streams in the arid region, are graphically portrayed in the following excerpt from a report made in 1921 by L. M. Lawson, M. Am. Soc. C. E., at that time Superintendent of the Rio Grande Project:

"In the relatively short time of five years since the completion of the Elephant Butte Dam, it has served the two extreme requirements, namely, that of storage and that of flood control. The crop produced during the period of 1917 and 1918, when practically all the water used for irrigation was drawn from storage, had a value in excess of the cost of the dam. The flood damage which would have resulted from the uncontrolled discharge of the flood of 1920, through the valleys of the Rio Grande above and below El Paso, would have reached an amount also largely in excess of the cost of the structure."

On the other hand, the fact that flood protection cannot in every case be expected to be provided by the reservoirs necessary for irrigation, is demonstrated by conditions on the Middle Rio Grande Conservancy District. This District comprises lands in the Rio Grande Valley between the upper end of the Elephant Butte Reservoir and the Colorado-New Mexico State line, and has been organized in order to solve in an orderly and co-ordinated manner the particularly acute problems of flood control, drainage, and irrigation, which must be solved to permit the fullest development of that part of the Rio Grande Valley. Here, the engineers have reached the tentative conclusion that dependence will have to be placed largely, if not exclusively, on levees for flood protection.

The construction of the Roosevelt Dam on the Salt River, in Arizona, has greatly improved flood conditions down stream, and the recent completion by the U. S. Bureau of Reclamation of the American Falls Reservoir on Snake River, Idaho, with a storage capacity of 1 700 000 acre-ft., has put an end to floods lower down.

An outstanding example of the possibilities of combining flood control and irrigation in streams of the arid region is the proposed Boulder Canyon Reservoir on the Colorado River. Here, the investigations of the Bureau of Reclamation engineers have demonstrated that by the construction of a dam in a narrow canyon raising the water surface 550 ft., a reservoir with a capacity of 26 000 000 acre-ft. may be formed, which would permit the utilization of the entire flow of the stream for irrigation and, in addition, would provide ample reserve capacity to regulate its floods to any desired extent. This would furnish flood protection for a number of irrigated valleys in the river bottoms between Needles, Calif., and Yuma, Ariz., and definitely remove the ever-present threat of disaster now hanging over Imperial Valley. This river is kept out of this basin now by means of levees along its north bank, which turn it south into the Pacific Ocean, but water is carried from it by a canal which runs through Mexico into the Valley, where 400 000 acres are now irrigated and 700 000 acres can be irrigated. Sixty thousand people live on the irrigated lands of this Valley, watered from the Colorado, with all their homes

below sea level and from 100 to 200 ft. below the level of the river where it crosses the International Boundary.

The Valley has once been threatened by destruction through inundation. For a year the whole volume of the Colorado poured into this basin, flooding farms, washing away houses, and doing millions of dollars of damage. Now, with the growing use of water along tributary streams, and the extension of its use in Mexico, further extension of irrigation is stopped, and the farms of the Valley are menaced by irreparable loss through drought. In September, 1924, less than one-third of the water needed by irrigators came down the river. There had been dangerous floods a few months before, followed by this devastating drought, that in two weeks caused a loss of $6 000 000 to the farmers.

The average flow of the Colorado River for a whole calendar year is about 16 000 000 acre-ft. This reservoir will hold, therefore, the entire discharge of the river for a year and a half. The great floods which now come down in the spring when the snows are melting will be caught here and held back, to be released later when water is needed to irrigate parched fields. No water will flow over the dam; all that goes down the stream will be discharged through its regulating gates which will open into tunnels that pass around the end of the dam, and will be cut through the towering cliffs between which the dam will be built.

POWER AS AFFECTING FLOOD CONTROL

F. W. SCHEIDENHELM,* M. AM. SOC. C. E.

Power development and flood control are so often incompatible that, from the standpoint of power development, one more readily states the subject of this paper in the inverted order, namely, flood control as affecting power development.

As a practical matter, consideration of power development in connection with flood control presupposes the use of reservoirs for such purposes. Some proponents of flood control conceive of benefits to power development as one of the economic justifications for control of floods by means of reservoirs. Proponents of water power development, on the other hand, are disposed to have misgivings whenever it is proposed to couple the use of storage capacity for flood control with that for power development. However, the two uses are not intrinsically, even though frequently, antagonistic, and it behooves one to analyze more deeply in order properly to appraise the effect of the one on the other.

For the purposes of this paper, the term, "floods", is assumed to include only those high waters which necessitate control in order to prevent damage.

Further, the writer assumes that the control of Mississippi River floods solely by means of reservoirs is impracticable. This subject has been discussed in some detail in certain other papers of this Symposium.

To the extent that reservoir sites may be available below Cairo, Ill., such reservoirs are not likely to be of great value for power development. The reservoirs could be filled safely only if and when important floods required the temporary storage of flood-crest water and as soon as practicable thereafter would have to be emptied so as to be ready to receive the water of the next flood crest, which might occur during the very same season or might not occur for several years.

The heads, where sufficiently great, are likely to be so variable and the water supply so intermittent that the possibilities for power installations in connection with Lower Mississippi flood-control reservoirs in general do not seem attractive. Certainly one could not reasonably expect power developments to contribute toward the cost of such reservoirs.

In connection with the use of levees, spillways, or by-passes for Mississippi flood control, power development is not a consideration. Here, too, heads would not be sufficient, nor is any head available for a time sufficiently long.

This paper, therefore, must be concerned mainly with reservoirs located on the tributaries of the Mississippi River and particularly in the upper portions of the drainage areas of those tributaries, for it is primarily at the headwaters that the fall occurs and power must be developed. Unfortunately, it is in those very areas that storage reservoirs are least effective as regards the control of floods on the Mississippi proper.

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The relationship existing between power development and flood control in respect to such reservoirs will be considered in the following categories:

I.—Complete Regulation by Storage.
II.—Partial Regulation by Storage.
(1) Reservoirs Solely for Power Development.
(2) Reservoirs Solely for Flood Control.
(3) Reservoirs for Joint Power Development and Flood Control.

I.—Complete Regulation by Storage

At the extreme of favorable relationship between power and flood control is the situation existing when a stream is under complete regulation or control by storage. By "complete regulation", from the standpoints of power development and flood control, is herein meant approximate equalization of run-off as of a given point on a given stream, for instance, to the extent of causing approximately the same aggregate run-off to occur month after month, but with variation in flow, for power purposes, within the months. Such regulation may be the work of man or of Nature. However, on any stream with as much as several thousand square miles of drainage area, complete regulation by artificial storage is not likely to be attempted for power purposes alone.

Whether the regulation be artificial or natural, and despite any utilization of the regulated run-off for power development, there can be practically no high stages of importance on such a stream save those due to ice gorges. This is true even though the stream flow be artificially regulated so as to yield little or no power at certain hours of the day, or days of a week, or weeks or seasons of a year, and correspondingly greater power during other hours, days, weeks, or seasons. In general, it is economically impracticable to install machinery of a capacity sufficient to utilize power from stream flow at such a rate as to produce a damaging flood wave.

The outstanding example of practically complete regulation is, of course, the St. Lawrence River, with its flow regulated by the Great Lakes. If one eliminates from consideration the "high water" caused by ice gorges, the St. Lawrence simply has no flood problem. The status would not be materially different were there artificial regulation for power purposes located, say, at the head of Niagara River, and were such regulation to release at times materially more than the normal flow.

The effect of the complete regulation of a tributary on a larger stream into which it discharges, constitutes a separate story. Generally, as regards flood control, that effect is favorable. Thus, the storing of water, especially of local flood run-off, cannot be otherwise than beneficial. Again, stored water on completely regulated head-waters of tributaries of the Mississippi would normally be released to offset the deficiencies in natural flow occurring either in mid-winter (for example, in the North), or in late summer or autumn (say, in the West, or in western regions of milder climate). In neither event can the water thus released from storage aggravate any major floods of the Mississippi, for apparently without exception these occur in the spring.

Whether complete regulation is feasible, whether it is economically warranted, and, if warranted, what proportions of cost should be borne by the respective beneficiaries thereof, are questions to be determined for each case, in the light of the facts of that case.

II.—Partial Regulation by Storage

Since the very question of the extent to which flood control is affected by power development implies some regulation by reservoirs, the category of partial regulation includes all pertinent cases other than those involving complete regulation.

(1) *Reservoirs Solely for Power Purposes.*—From the short range standpoint, that is, as concerns areas down stream from the reservoirs, but not too far (say, within several hundred miles), such power reservoirs can only be beneficial in respect to flood control. This benefit, however, may be negligible and certainly is not to be relied on as adequate protection against floods.

If the storage capacity of a reservoir is utilized, then, axiomatically, the storage is drawn upon either when the natural stream flow is relatively low, or in advance and in anticipation of a flood. Conversely, the reservoir is usually refilled when the stream flow is above normal, preferably when the stream is in flood. The result can only be a tendency to reduce flood crests in near-by down-stream areas; certainly, no harm can result.

Such beneficial effect is increased if the storage capacity is sufficient to absorb the entire flood and if, at the same time, other power capacity is available for supplying market demands, so that during the flood the power use at the reservoir may be temporarily suspended in whole or in part. Indeed, if such other power capacity is "hydro" and is located at power stations on streams simultaneously in flood, the benefit from the standpoint of flood control would be accompanied by benefit from the standpoint of power development as well.

Even if the storage capacity of a reservoir be not utilized by drawing upon it, there is generally an absorption of flood peaks. Such absorption is not necessarily important in quantity, but is bound to be beneficial in quality. The absorption takes place without exception wherever the dam or spillway is not controlled by gates or siphons. It results from the fact that, as the flood discharge increases, the depth of flow over the spillway crest increases. This, in turn, necessitates the storing of water over the reservoir surface area to a depth corresponding to the increase in depth of over-flow. The water thus stored must necessarily include the run-off at the very peak of the flood. The result is to decrease the peak of the flood flow from the dam as compared with the peak of the flow into the reservoir.

The smaller the drainage area above the dam and the larger the surface of the reservoir, the greater is such flood-peak absorption. As an extreme there may be cited a drainage area of about 58 sq. miles proposed to be controlled by a storage dam for a power project in West Virginia, with which the writer has had considerable to do. There the reservoir area of about 6 450

acres will be relatively so great that, even assuming the reservoir to be filled at the beginning of the probable maximum flood, the actual flood peak passing over a spillway only 100 ft. long, would be hardly 5% of the natural peak at that point. A case applying to a larger drainage area, namely, 1 044 sq. miles, is that cited* by E. H. Sargent, M. Am. Soc. C. E., for the Sacandaga Reservoir with an area of about 26 000 acres, under construction on a tributary of the Hudson River. A flood of the same unit intensity and graph as the Miami River flood of 1913 apparently would have its peak reduced by one-half in passing through the Sacandaga Reservoir and over a spillway assumed to be 600 ft. long. In this case, too, the reservoir was assumed to have been full at the beginning of the flood.

The flood-peak water thus absorbed is retained only temporarily and upon subsidence of the flood is released gradually, but with the result of prolonging the period of high water.

From the long-range standpoint, that is, as concerns 1 000, or more, miles down stream from the reservoir, the effect may be beneficial, or it may be quite the contrary. If one considers the Mississippi River, and the fact that its flood peaks are due to varying coincidences of flood peaks from its several principal tributaries, it is clear that the retardation of flood-peak run-off from a given tributary might prevent the arrival of that peak run-off at a critical junction point, such as Cairo, at a time when it would aggravate a situation arising from synchronous arrivals of floods from other tributaries.

Conversely, the tremendous drainage area of the Mississippi and the long distances of flood travel on its tributaries make equally possible the circumstance that if a given head-water flood peak had not been disturbed or retarded, its water would have reached the critical junction head of the main coincident flood flows, whereas the retardation in fact aggravates the synchronism and hence the resultant Mississippi flood peak.

As regards the Mississippi, therefore, it seems fair to state that over a long period of time the beneficial and the harmful effects of local flood-peak absorption would offset each other. Ample evidence is available to show that the magnitude of effect in either direction can only be negligible.

Finally, it is well to bear in mind that an operating procedure which would involve drawing down a power reservoir in advance of the flood season, so as to afford some additional flood control, is likely to be detrimental to power development. It would involve reduction in head and, hence, in power and, should the anticipated flood not occur, there might be no opportunity to refill the depleted reservoir.

(2) *Reservoirs Solely for Flood Control.*—As to method of operation or purpose these may be classified, as follows:

(*a*) Reservoirs intended to prevent completely any flow past the dam until the reservoirs overflow.

The object is to postpone, completely if possible, the discharge of flood waters until after the flood flows from drainage areas farther down stream have passed the region to be protected. Such a reservoir may be intended

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 973.

to protect areas at considerable distances down stream, for instance, for the protection of Pittsburgh, at the junction of the Monongahela and Allegheny Rivers to form the Ohio River, in Western Pennsylvania, there has been proposed a system of reservoirs which, among others, would include reservoirs in West Virginia on the head-waters of the Monongahela River.

(*b*) Reservoirs or basins intended to detain the flood discharge, or, more accurately, to lessen the rate of flood discharge to such as may be carried safely by the stream channels through the areas to be protected.

Detention reservoirs are intended to protect areas relatively close to the reservoirs. The outstanding example is the system of the Miami Conservancy District, intended to protect the Cities of Dayton and Hamilton, Ohio, and certain smaller towns.

A reservoir of either type intended solely for flood control in general cannot reliably fulfill its function unless it be emptied as soon as practicable after a flood and thus made ready for the next flood. (Detention reservoirs cause this to take place automatically.) The result is that the flood flows completely withheld, or merely retarded, as the case may be, are released long before the natural stream flow reaches a stage materially below normal. In other words, the flood flows can not, must not, be utilized to increase natural flows during low-water periods. Consequently, benefits to power developments are necessarily incidental and, generally, are minor in amount.

It is conceivable that in regions where spring run-off and all floods are in large part due to the melting of snow, as in the Rocky Mountains, a flood-control reservoir might safely store for power or other purposes the last installments of spring run-off, once the melting of snow has been completed or has reached a predetermined stage.

(3) *Reservoirs for Joint Power Development and Flood Control.*—There have been various proposals for the construction of such reservoirs, some of these proposals having been made under the auspices of the Federal Power Commission. However, the writer knows of no actual instance of construction.

A clause for this purpose, proposed by the Federal Power Commission to be embodied in a license for a series of water power developments on a tributary of the Ohio River, reads as follows:

> "If at any time any agency shall contribute funds to provide additional reservoir capacity for the purpose of storage of water for flood control, the licensee shall co-operate with such agency by constructing or permitting to be constructed, to the extent of the funds so contributed, additional reservoirs or additions to existing reservoirs as may be agreed between the licensee and such agency; and the licensee shall so operate its project works as to hold, or permit to be held, in reserve for flood control the storage capacity provided by funds so contributed."

The prospective licensee indicated that the clause would be satisfactory.

Usually, it is contemplated that the flood-control reservoir capacity be literally superimposed upon that intended for power uses. The object is to take advantage of favorable reservoir sites, especially in regions where favorable sites are scarce. Presumably, there are situations where the combination of the two objects can be attained at less expenditure for reservoir capacity

than would result from the construction of separate reservoirs for power and for flood control.

The combined reservoirs must in effect be considered and operated as if they were separate. The danger is that the desires, or the needs, for power will tend to hold water too long in that part of the reservoir allocated to flood control. However, the penalty of mal-operation of the flood-control component of the reservoir is so great that power development ought not to be benefited more than it would be if the storage capacity allocated to flood control were actually in a separate reservoir.

Such a combination in a single reservoir would have the incidental effect of making impossible the development of power from the head which is represented by the depth of the flood-control component of the reservoir.

This subject is too involved for detailed treatment within the limits of this paper. However, in the light of the foregoing the following conclusions are offered:

1.—As regards the flood control of a given stream, power developments involving reservoirs on that stream generally have a beneficial effect and in especially favorable cases may have important beneficial effect.

2.—On streams which are under complete regulation by storage, that is, where approximate equalization of flow is attained, the requirements of both power development and of flood control on that stream may be met adequately and without conflict. Complete regulation is a desideratum, even though not a necessity, from both standpoints. (However, in the case of the Mississippi River it seems certain that adequate flood control, let alone complete regulation, cannot be obtained solely by means of storage reservoirs. Even for streams with drainage areas as small as several thousand square miles, complete regulation artificially must remain a rarity.)

3.—In the cases of streams which are under only partial regulation by storage, power developments may have favorable or unfavorable effects from the standpoint of flood control; in the main, however, such effects of power development will tend to be beneficial.

4.—So far as flood control by means of reservoirs involving only partial regulation of a given stream is concerned, the benefits which power developments even on that particular stream would derive are, in general, so limited in amount and so uncertain in occurrence that power could not economically contribute any substantial financial aid toward flood control.

5.—As regards the flood control of the Mississippi River, power is likely to be a minor consideration, both as to effects of power reservoirs in reducing Mississippi River floods and as to financial aid toward flood control.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

EMERGENCY DAM ON INNER NAVIGATION CANAL AT NEW ORLEANS, LOUISIANA

BY HENRY GOLDMARK,* M. AM. SOC. C. E.

TO BE PRESENTED DECEMBER 7, 1927

SYNOPSIS

This paper is devoted primarily to a description of the so-called "emergency dam" in the Inner Navigation Canal at New Orleans, La., an important harbor work, completed in 1922. Such "emergency dams" are designed to shut off the flow of water through a lock, such as may occur when one of the lock-gates is struck by a vessel and seriously injured.

As the dam at New Orleans differs radically in its design from those previously built and has the merit of being lower in first cost and requiring a smaller operating force, a description of the work should be of interest to engineers. The paper first enumerates certain protective devices for preventing accidents to the gates, such as special guard gates, power capstans, and electric towing locomotives for controlling vessels in transit, and chain fenders for bringing them to a stop if improperly handled. A brief reference is then made to some emergency dams previously built, especially those in the Sault Ste. Marie and Panama Canals. Finally, a full description is given of the New Orleans Dam.

This design may be called the "stop-log" type, since the flow of water is checked by a number of transverse girders spanning the lock chamber, placed one above the other, like the stop-logs in hydraulic power plants. These girders are lowered and raised by a hoisting engine carried on a revolving crane, similar in appearance to a railroad swing bridge. The several parts of the dam are described and illustrated.

NOTE.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in April, 1928.

* Cons. Engr., New York, N. Y.

Introduction

Although the risk of serious accidents in properly built and operated ship locks is very small, there are unquestionably certain dangers inherent in their use. Such accidents as do occur arise, in most cases, from injuries to the lock-gates when they are accidentally struck by vessels. Even if the damage to the gate is small, the repairs are difficult to make and certain to result in delays to traffic. If the gate is more seriously injured it is likely to be carried away by the rush of water; other gates may then be torn loose, and a heavy flow established from the upper to the lower pool.

Various safeguards are installed in locks to prevent vessels from striking the gates. The most common of these are duplicate lock-gates, the so-called "guard gates", placed above and below the ordinary operating gates to act as barriers to protect the latter from injury.

As a further measure of safety many large locks are fitted with power capstans for controlling vessels in transit. At Panama, however, all vessels are towed through the locks by special locomotives traveling on tracks laid on the lock walls. In this way the ships are kept under full control at all times.*

As an additional protection to the gates, heavy chain fenders stretched across the lock chambers have been installed in the approaches and upper chambers of the Panama Canal, and have also been in use for many years in a number of English harbors. When struck by a vessel, the chain pays off under a heavy braking strain, gradually bringing the ship to rest. At Panama, these fenders have proved their value in actual service.† In several instances, vessels, some of them of large size and moving at a fair rate of speed, were brought to a stop by the chain without injury to the vessel, fender, or lock-gate.

Besides such safeguards intended solely to prevent vessels from striking the gates, special devices have been installed in some locks for shutting off the flow of water if, in spite of these safeguards, one or more of the gates is seriously injured. However, such "emergency dams" are necessary only in exceptional cases. They are not required, for instance, in tidal locks, since in these there is slack-water at each period of high and low tide when the guard gates at the ends of the lock can be closed without trouble.

They are also superfluous where the pool above the lock is small, or where a series of locks is separated by short stretches of canal. In these cases the water above the lock can be readily drained off, so that a special dam is unnecessary.

When, however, the body of water above the lock is a large river or lake, unquestionably it should be possible to shut off the flow in case of an accident, to permit repairs to be made with reasonable promptness. In some cases it would be impracticable—in any event, very time-consuming—to improvise means for accomplishing this object after the accident had occurred.

* "Electrical and Mechanical Installations of the Panama Canal," by E. Schildhauer, M. Am. Soc. C. E., *Transactions*, Int. Eng. Cong., San Francisco, Calif., 1915 (The Panama Canal), Vol. II, No. 20.

† "Lock-Gates, Chain Fenders, and Lock Entrance Caisson," by Henry Goldmark, M. Am. Soc. C. E., *Transactions*, Int. Eng. Cong., San Francisco, 1915 (The Panama Canal), Vol. II, No. 17.

Special "emergency" dams have been installed, therefore, in the canals at Sault Ste. Marie and Panama, and in some other waterways. The latest of such dams is that built in the Inner Navigation Canal, at New Orleans, which is the special subject of this paper. A brief reference to some of the older ones, however, may be of interest.

Early Emergency Dams

As far as is known to the writer the first "movable" or "emergency" dam installed in connection with a lock was designed by the late Alfred Noble, Past-President, Am. Soc. C. E., for the United States Canal at Sault Ste. Marie, Mich., more than forty years ago. It was removed later when the canal was widened. Its general appearance was that of a railroad swing bridge revolving about a vertical pivot on the lock-wall. Along one of the arms a series of wicket girders was hinged by pins. After the dam had been turned across the lock, these girders were revolved in planes at right angles to the axis of the dam until their lower ends rested against a concrete sill. After the girders were thus put in place, the openings between them were successively closed by small wickets.

This gradual reduction in the cross-section of the flowing water is essential on account of the high velocity with which it moves. Such a principle is necessarily followed in all dams of whatever type.

At a later period, an emergency dam of quite similar design was built on the Canadian side of the St. Mary's River, also at Sault Ste. Marie. It is unique in having been actually tested under emergency conditions.

The accident, which occurred in June, 1909, was due to a vessel approaching the locks from below and striking the lower operating gates forcing them partly open and causing a heavy flow of water from the lock. At this time a vessel was in the lock and the upper gates were being closed. The heavy flow broke the fastenings of the upper and lower gates. Some of the leaves were torn entirely loose and others were badly damaged.

The vessel in the lock, the one in the lower approach, and a third one just above the lock were all carried away into the lower river, fortunately without loss of life. The flow of water was checked by closing the emergency dam, which was accomplished without serious difficulty. Figs. 1 to 4 show various phases of this accident.

In addition to these dams, which are all essentially of the same type, at least one other important structure of different design, but serving a similar purpose, should be mentioned. This is the "butterfly" dam on the Chicago Drainage Canal, which revolves about a vertical axis at one edge of the channel, the clear opening being 160 ft.

Panama Canal

When the plans for the Panama Canal were being matured, it was decided to adopt an emergency dam of the swing type with girders and wickets, quite similar to the dams at the "Soo". This decision followed the recommendations

of a Board of Army Engineers appointed some years before to study the general question of emergency dams.

The Panama dams,* six in all, are longer and heavier than any others. Great care was taken to make them perfect in all their details, so that their operation is smooth and rapid. The swing span is revolved and the girders and wickets are lowered and raised by separate electric motors.

New Orleans Inner Navigation Canal

This canal is situated in the lower part of the City of New Orleans and connects the Mississippi River and the tidal inlet known as Lake Pontchartrain.† Its primary purpose is to furnish an "inner harbor" of approximately constant depth, but it may ultimately provide an outlet to the Gulf of Mexico shorter and in other ways more desirable than the Lower Mississippi. The canal is 5 miles long, with a lock at its upper end close to the river in order to overcome the difference in elevation between the Mississippi and Lake Pontchartrain. Both sides of the canal between the lower end of the lock and the lake are to be lined with docks or piers, while it will also be possible to utilize the shores of Lake Pontchartrain for docking purposes. The canal has a bottom width of 150 ft. The lock is 75 ft. wide, with a depth of 30 ft. on the lock-sills and a maximum lift of 19 ft. (Fig. 5).

In case of serious accident to the lock-gates, a heavy flow of water would occur between the upper and lower levels, the maximum volume being computed to be in excess of 70 000 sec-ft.—about equal to one-third the total flow over the American and Canadian Falls, at Niagara. As a large part of the city below the lock, including important railroad yards, would be flooded in case of an accident, it was vital that the dam should be reliable beyond question and rapid in operation.

The emergency dams on the Panama Canal meet these requirements, but their cost was very high and a large number of attendants is required for their operation. It was thought possible that some other form of construction might be devised, which would be as efficient as the Panama dams but lower in first cost, and require a smaller operating force.

After several types had been considered, in addition to the standard swing-bridge design, it was suggested by Mr. R. O. Comer, Designing Engineer of the New Orleans Port Commission, that it might be possible to shut off the flow of water by a series of transverse girders, sliding in grooves in the side-walls, like the "stop-logs" used in power plants. It would be necessary, of course, to provide suitable structural work and machinery for placing and removing the girders when operating the dam.

A careful study showed that a dam of this kind could be built—one that would be simple and reliable and cost decidedly less than the standard form used elsewhere. It was therefore adopted for construction (Fig. 6). At that time the stop-log type of dam was believed to be entirely novel in lock con-

* The Panama dams are described in detail in a paper by their designer, T. B. Mönniche, M. Am. Soc. C. E., and published in *Transactions*, Inter. Eng. Cong., San Francisco, Calif., 1915 ("The Panama Canal"), Vol. II, No. 18.

† A brief description of the entire Inner Harbor Navigation Canal, by the writer, may be found in *Proceedings*, Brooklyn Engrs. Club, Vol. XXV, Pt. I (October, 1926), pp. 36–49.

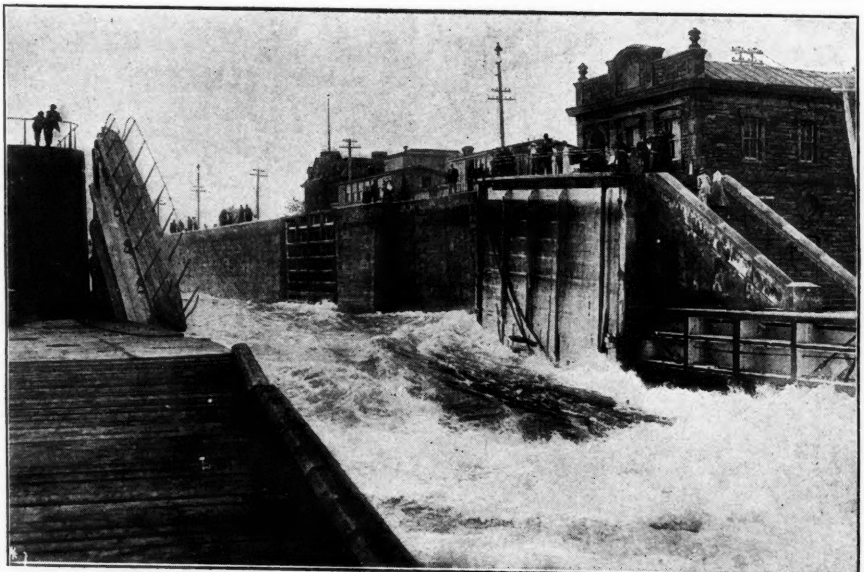

FIG. 1.—ACCIDENT TO SAULT STE. MARIE LOCK, SHOWING LOWER GATES SHORTLY AFTER THEY HAD BEEN HIT BY VESSEL.

FIG. 2.—DAMAGED SAULT STE. MARIE LOCK, LOOKING EAST FROM UPPER END OF LOCK, SHOWING RECESSES FOR UPPER GATES TORN AWAY BY FLOW.

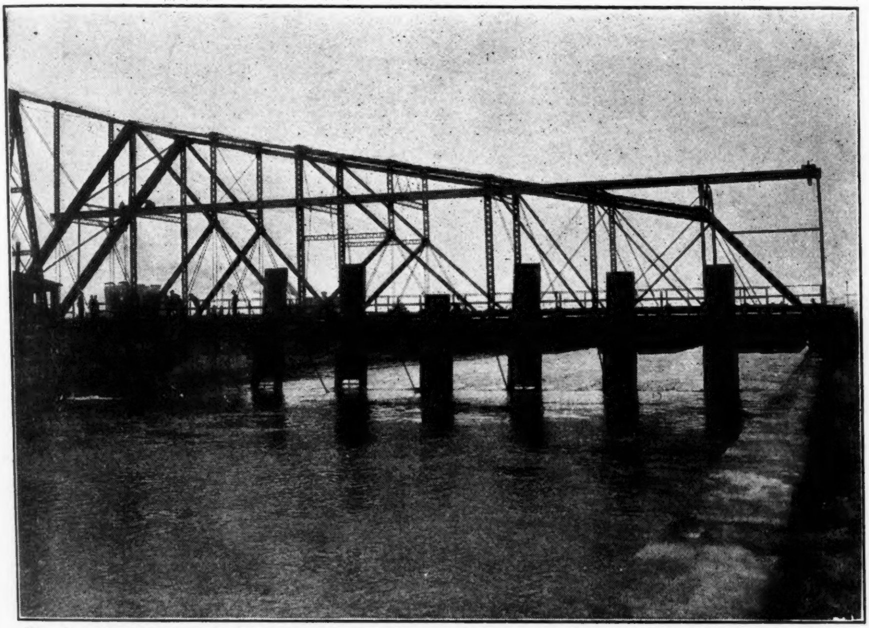

FIG. 3.—OPERATION OF EMERGENCY DAM, LOOKING DOWN STREAM. FIRST WICKETS BEING LOWERED INTO PLACE, REMAINDER SUSPENDED IN AIR BELOW BRIDGE.

FIG. 4.—CLOSURE OF EMERGENCY DAM ALMOST COMPLETED, SHOWING LEAKAGE WHICH COULD NOT BE AVOIDED.

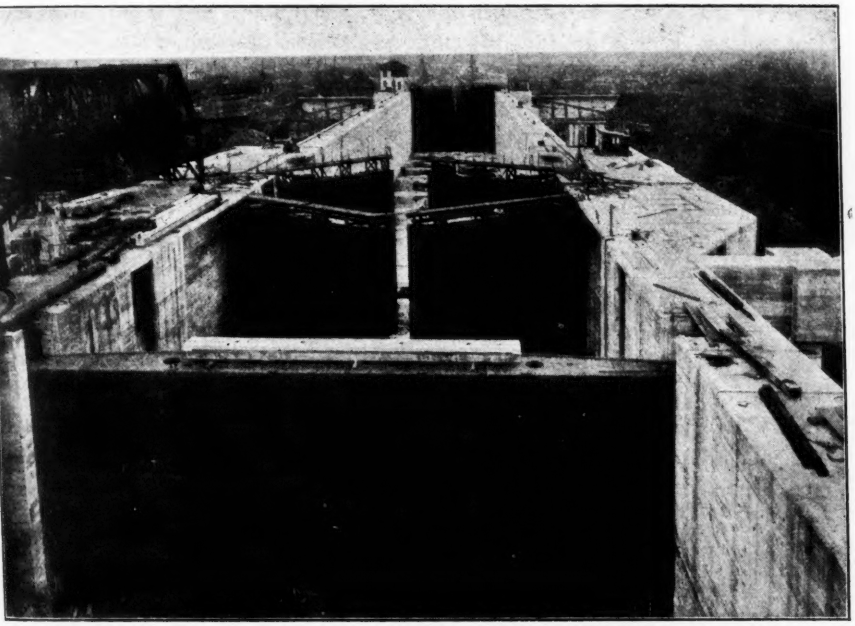

FIG. 5.—GENERAL VIEW, NEW ORLEANS INNER NAVIGATION CANAL LOCK, WITH EMERGENCY DAM IN FOREGROUND.

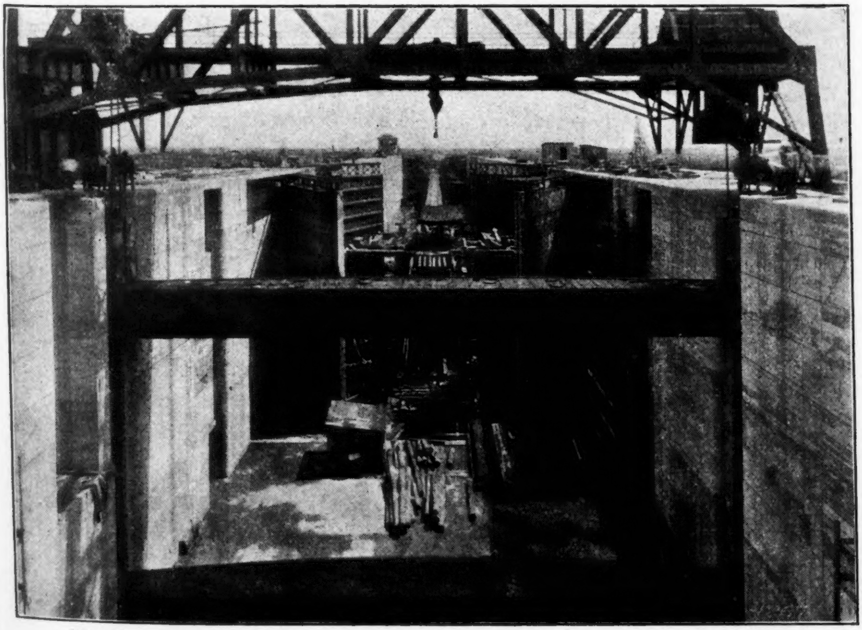

FIG. 6.—OPERATION OF EMERGENCY DAM, LOWERING STOP-LOG GIRDER FROM REVOLVING CRANE.

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struction, but it was afterward found that at least one dam of the kind had been built in Canada, on the Trent Canal, although on a much smaller scale, and with entirely different details.

General Features—New Orleans Dam

The general arrangement of the New Orleans Dam is given in Fig. 7, showing the ground plan, cross-sections, and end elevation. Various details of the structural work and mechanical parts are shown in other diagrams and photographs.

It will be seen that the stop-log girders are handled by a huge revolving crane or swing span, with its pivot on one of the lock-walls. Except when in use for closing the lock-opening, the girders rest on the top of the back-fill with their axes radial (Fig. 7). From these positions they are raised by the handling machinery, the swing span is revolved, and the girders are then placed one above the other, with their ends in the recesses in the side walls. This operation is reversed when they are being removed after they have served their purpose.

While the dam was built primarily for checking the flow of water in case of accident, the same girders are also used to form a coffer-dam at the upper or lower end of the lock when it is desired to pump it out completely and expose all the lock-gates for inspection, painting, and repair. When the girders are to be used at the lower end, they are lifted from their seats in the storage yard by the revolving crane and lowered into the water in the position indicated by the dotted outline in Fig. 7. They are then floated through the locks. Of course, it was necessary to add vertical seats for the girders at the lower end of the lock, and a simple form of hoist was also installed for raising and lowering them. These hoists are operated by two of the power capstans ordinarily used for handling vessels in transit. It will be seen that the additional expense entailed was quite moderate.

In order to give a clearer understanding of the construction and operation of the dam, it has seemed best to describe its various parts separately. They are:

I.—The Stop-Log Girders.
II.—The Swing Span, or Revolving Crane, with the Mechanism for Turning It.
III.—The Hoisting Machinery for Handling the Girders.

I.—The Stop-Log Girders

The girders are of box-form with plate-webs and side-plates (Fig. 8). There are eight girders in all. Six of them, which are used at the lower as well as the upper end of the lock, are designed so as to float when free from water. The other two are not adapted for floating. All the girders are 84 ft. long over-all, and 7 ft. 9½ in. wide horizontally at the center of the span. Their depth is about 6 ft. The weight of the heavy girders in the dry is

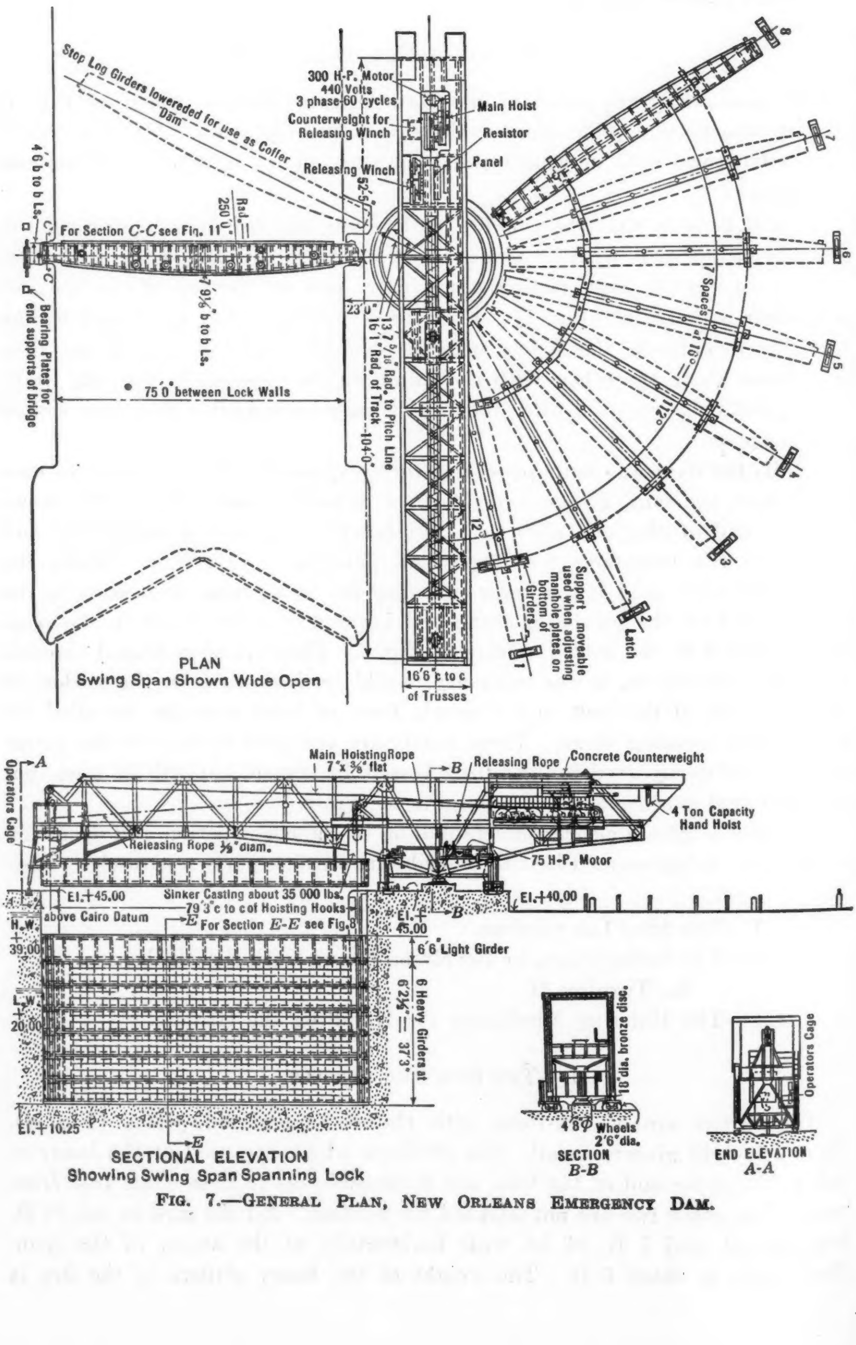

FIG. 7.—GENERAL PLAN, NEW ORLEANS EMERGENCY DAM.

174 000 lb. In these, both the web and side-plates are strengthened by closely spaced channel stiffeners.

When the lock is pumped out the girders are under maximum hydrostatic pressure, the water extending to the top on the upper face with no water on the lower side. While this maximum head will act only on the bottom girder, it was thought best to make all the "heavy" girders of identical construction to guard against errors in placing them.

When the girders are being lowered into place under emergency conditions —that is, when there is a flow of water through the lock—the horizontal pressures are a combination of the velocity head due to the flow and the static head at the various levels. The computation of the loads due to these conditions is quite involved. In this case, too, the worst possible conditions have been assumed.

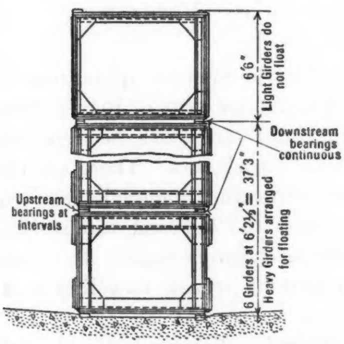

FIG. 8.—VERTICAL SECTION THROUGH STOP-LOG GIRDERS, SECTION *E-E* OF FIG. 7.

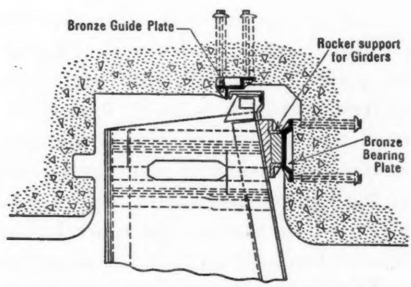

FIG. 9.—SECTION THROUGH SLOT IN WALL, SHOWING BEARING AND GUIDE-PLATES.

As noted, most of the girders are arranged for flotation, but when they are being lowered, they must be filled with water, to insure their sinking. This made it necessary to supply these girders with a sufficient number of openings fitted with adjustable covers. These openings also allow the water to flow out of the girders at a proper rate when they are being raised, thus guarding against an over-load in the hoisting machinery and excessive stresses in the stop-logs.

In operation, the heavy horizontal pressure exerted against the up-stream side of the girders produces frictional resistances between the bearings and the slots in the side walls. Much study was given to the proper design of these bearings, shown in Fig. 9. While roller bearings would have reduced the friction materially, it was thought best to avoid devices that were complicated and subject to corrosion in the brackish water of the lock. It would also have been necessary to provide separate means to insure water-tightness. The fixed bearings in the wall, therefore, were made continuous from top to bottom. They are of bronze at the upper, and of cast iron at the lower, end of the lock. The bearing plates on the several girders are of steel and are made as rockers revolving slightly about their vertical axes, to adjust themselves to

inequalities from errors in workmanship or from deflections of the girders under their loads. The fitting of these bearings has proved to be excellent; there is no leakage even when the lock is pumped out and the girder is under full hydrostatic head.

Water-tightness along the horizontal joints between the several girders and also between the lowest girder and the bottom of the lock chamber is insured by continuous metallic bearings along the down-stream flanges. These joints also have proved to be tight. At their ends, the girders are fitted with vertical steel channels. Similarly, continuous bronze guide-plates are fastened to the face of the concrete.

This description covers all parts of the stop-log girders, except some mechanical attachments, which may be described more properly in connection with the hoisting machinery.

II.—Swing Span, or Revolving Crane, with the Mechanism for Turning It

The structural work of the revolving crane has the general appearance of a railroad swing bridge with unequal arms. The longer arm is 104 ft. from the pivot to the end, the shorter, 52 ft. 5 in.; and the width between the centers of trusses is 16 ft. 6 in. All connections are riveted. There are the usual top and bottom lateral systems with transverse and diagonal members, besides the secondary structural parts that support the hoisting mechanism. At the end of the shorter arm is a heavy concrete counterweight. The span is carried in part on the central pivot and in part on wheels traveling on a circular track 32 ft. 2 in. in diameter.

The details of the pivot are shown in Fig. 10 and a diagrammatic view of the device for attaching the hoisting hooks in Fig. 11. The pivot has an upper disk of phosphor bronze, 18 in. in diameter, which turns with the span; and a lower disk of the same size made of forged steel, which is secured to a steel casting resting on the concrete (Fig. 10). The wheels that travel on the circular track are eight in number. Their diameters are 2 ft. 6 in. and the width of face is 5½ in. They have cast-steel centers and rolled-steel tires. The axles are 7 in. in diameter and turn in journals on four truck bodies of cast steel bolted to the lower side of the bridge truss.

The swing span is turned about the pivot by a system of gears operated by a 75 h. p., electric motor, the type of mechanism being that commonly used in draw-bridge machinery. The principal details are shown in Fig. 12.

Stresses in Truss Members and Reactions on Pivot and Truck Wheels.—The loads carried by the truss are: (1) The dead load, which consists of the structural steel, the machinery, the concrete counterweight, and various minor parts; and (2) the live load, that is, the heaviest stop-log girder.

The calculation of the stresses and reactions presented some interesting features differing from those met in similar structures. It was thought worth while, therefore, to include in this paper, a tabular statement of the stresses and reactions which occur under various cases of loading (Table 1) and a diagram (Fig. 13), giving an outline of the truss, together with the panel loads

In Table 1 the make-up of the various members is given in Column (10). The size of the concrete counterweight (277 000 lb.) was computed to balance the whole dead load and one-half the live load so that the reaction against the truck at Panel Point 9 (Fig. 13) when the dead load alone is being carried should be exactly the same as the reaction at Panel Point 7 when the trusses also carry the live load. In the case under Column (5), Table 1, all

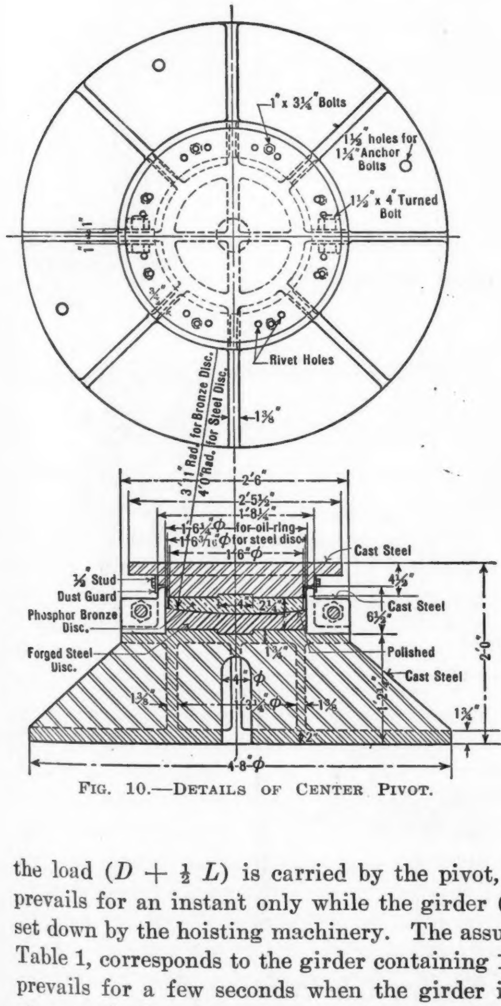

FIG. 10.—DETAILS OF CENTER PIVOT.

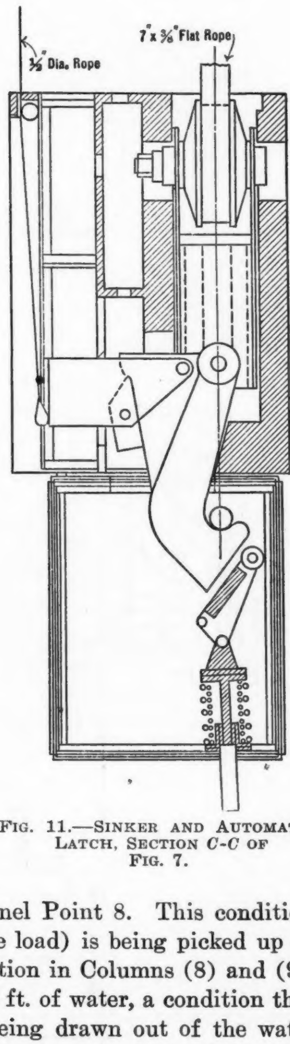

FIG. 11.—SINKER AND AUTOMATIC LATCH, SECTION C-C OF FIG. 7.

the load $(D + \frac{1}{2} L)$ is carried by the pivot, Panel Point 8. This condition prevails for an instant only while the girder (live load) is being picked up or set down by the hoisting machinery. The assumption in Columns (8) and (9), Table 1, corresponds to the girder containing 1.43 ft. of water, a condition that prevails for a few seconds when the girder is being drawn out of the water at a speed of 30 ft. per min. The details of the various loads carried by the trusses are given in Table 2.

In the computations, both arms of the truss were taken as cantilevers, as this condition will prevail practically at all times. It was deemed best, however, to fix the end of the long arm vertically when the truss has been swung

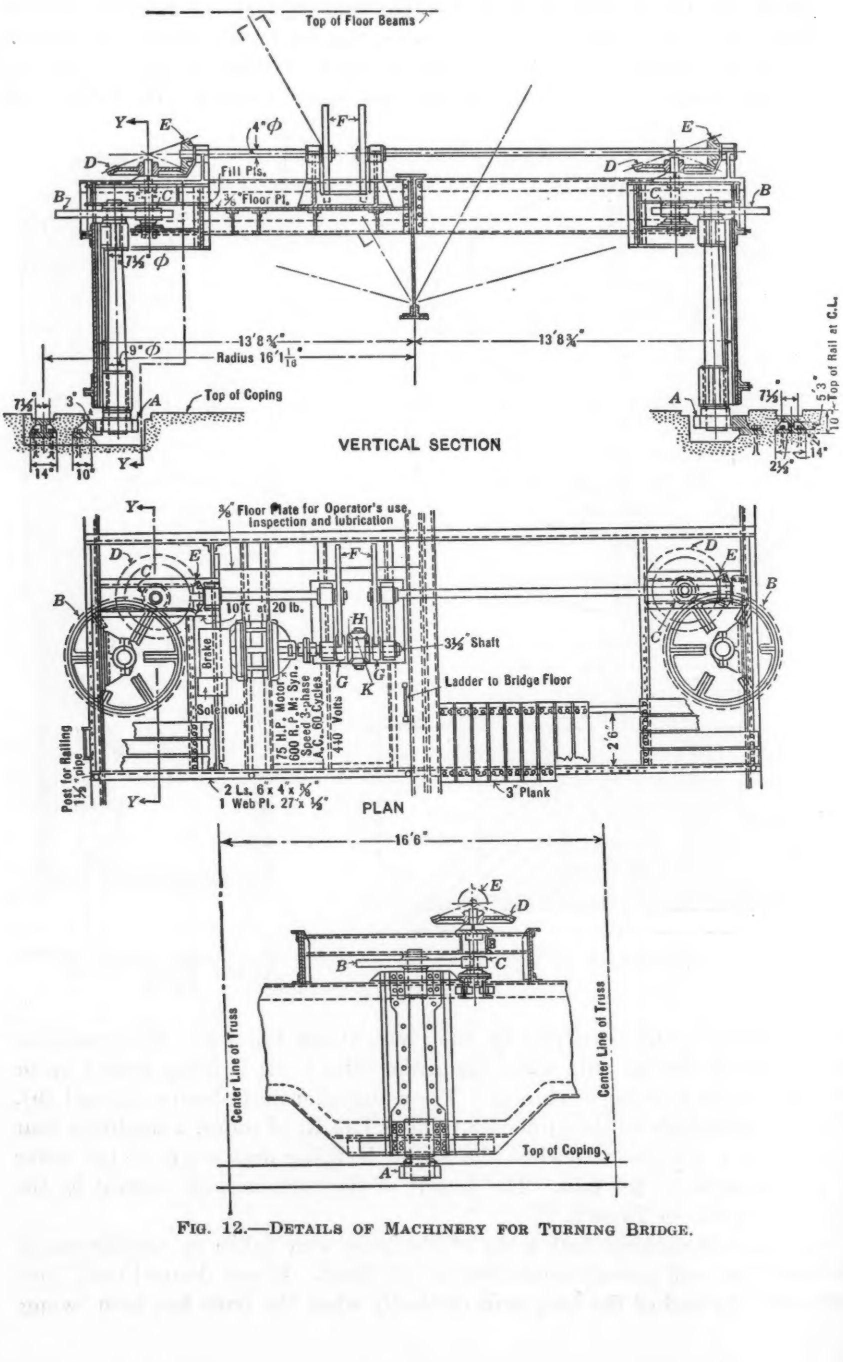

FIG. 12.—DETAILS OF MACHINERY FOR TURNING BRIDGE.

across the lock and the girders are being raised or lowered. A braced framework therefore was attached to the truss at Panel Point 0, so that the end of the truss can be slightly raised by a screw-jack attached to the framing. Under these conditions, a small part of the dead and live loads is supported by the side wall, but the stresses in the truss are only slightly changed (Fig. 7).

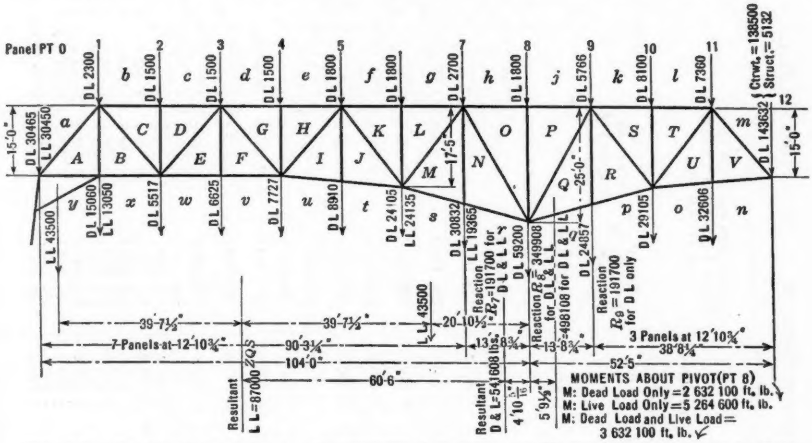

FIG. 13.—DIAGRAM OF TRUSS. GOVERNING STRESSES IN VARIOUS MEMBERS FOR ONE TRUSS ARE SHOWN IN TABLE 1.

III.—THE HOISTING MACHINERY FOR HANDLING THE GIRDERS

The hoisting machinery includes not only the hoisting engine proper, but also the cables with their sheaves, the so-called "sinker castings," the hooks for lifting the girders, and certain devices for detaching the hooks from the girders, after the latter have been lowered into place. These last devices are important features of the dam. Their design proved to be quite difficult.

The general arrangement of the machinery is given in Fig. 7, while Fig. 14 shows the hoisting engine on a larger scale, as well as an outline of the electric apparatus connected with it. It also shows a small hand-operated winding engine forming a part of the machinery used in detaching the hooks. The details of these two engines were developed by the contractor for the hoisting machinery.

The two main hoisting cables consist of 7 in. by ⅝-in., flat, steel ropes. One end of each rope is wound about a separate drum in the hoisting engine, the two drums being connected by gearing and driven by a single 300 h. p. electric motor. The other end of the rope passes around three 30-in. sheaves, multiplying the lifting power threefold, and connects with the massive hook from which one end of the girder is suspended. The journals for two of these sheaves, at each end of the truss, are bolted firmly to the structural work, while the third sheave moves up and down as the hook is raised or lowered. A bight of the flat rope makes a half-turn about this sheave, while the end of the rope is fastened to a clevis through which the axle of the sheave passes. From the same axle a riveted structural frame is hung, to which the hook for suspending the girders is connected. (See Figs. 11 and 15.)

TABLE 1.—STRESSES IN ONE TRUSS.*

(t = tensile stress; c = compressive stress; n = net cross-section; gr = gross cross-section.)

	Member. (1)	Cross-section, in square inches. (2)	Stresses from dead loads, in pounds. (3)	Stresses from live load, in pounds = 87 000. (4)	Stresses from one-half live load, in pounds = 43 500. (5)	Total maximum stresses, in pounds. (6)	Unit stress, in pounds per square inch. (7)	STRESSES FROM DEAD LOAD PLUS A LIVE LOAD OF 110 000 LB.: Total stresses, in pounds. (8)	STRESSES FROM DEAD LOAD PLUS A LIVE LOAD OF 110 000 LB.: Unit stress, in pounds per square inch. (9)	Section adopted. (10)
End post.	*a A*	19.2 *n*	39 900 *t*	39 900 *t*	19 950 *t*	79 800 *t*	4 150 *t*	90 570 *t*	4 700 *t*	Two 12-in. × 25-lb. channels. One 22 × ⅜-in. cover plate.
Upper chord.	*b C* and *z D*	19.2 *n*	67 330 *t*	63 600 *t*	31 800 *t*	130 930 *t*	6 820 *t*	148 100 *t*	7 700 *t*	Two 12-in. × 25-lb. channels. One 22 × ⅜-in. cover plate.
Upper chord.	*d G* and *e H*	22.4 *n*	168 560 *t*	134 850 *t*	67 425 *t*	307 010 *t*	13 700 *t*	339 760 *t*	15 200 *t*	Two 12-in. × 25-lb. channels. One 22 × 9/16-in. cover plate.
Upper chord.	*f K* and *g L*	33.4 *n*	260 000 *t*	183 600 *t*	91 800 *t*	443 600 *t*	13 280 *t*	493 200 *t*	14 800 *t*	Two 12-in. × 40-lb. channels. One 22 × ¾-in. cover plate.
Upper chord.	*h O* and *j P*	30.2 *n*	314 000 *t*	106 000 *t*	106 000 *t*	420 000 *t*	13 900 *t*	420 000 *t*	13 900 *t*	Two 12-in. × 40 lb. channels. One 22 × ½-in. cover plate.
Upper chord.	*k S* and *l T*	19.2 *n*	239 200 *t*	0	0	239 200 *t*	12 460 *t*	239 200 *t*	12 460 *t*	Two 12-in. × 25-lb. channels. One 22 × ⅜-in. cover plate.
End post.	*V m*	19.2 *n*	174 200 *t*	0	0	174 200 *t*	9 070 *t*	174 200	9 070 *t*	Two 12-in. × 25-lb. channels. One 22 × ⅜-in. cover plate.
Lower chord.	*A y* and *B x*	19.8 *gr*	26 200 *c*	26 180 *c*	13 090 *c*	52 380 *c*	2 650 *c*	59 470 *c*	3 000 *c*	Two 15-in. × 33-lb. channels.
Lower chord.	*E w* and *F u*	19.8 *gr*	114 500 *c*	101 000 *c*	50 500 *c*	215 500 *c*	10 880 *c*	242 800 *c*	12 250 *c*	Two 15-in. × 33-lb. channels.
Lower chord.	*I a* and *J t*	34.8 *gr*	214 150 *c*	163 200 *c*	81 600 *c*	377 350 *c*	10 840 *c*	421 450 *c*	12 110 *c*	Two 15-in. × 33-lb. channels. Two 12 × ⅝-in. plates.
Lower chord.	*M s* and *N r*	45.8 *gr*	291 700 *c*	200 100 *c*	100 050 *c*	491 750 *c*	10 740 *c*	545 800 *c*	11 910 *c*	Two 15-in. × 55-lb. channels. Two 12 × 9/16-in. plates.
Lower chord.	*Q q* and *R p*	32.2 *gr*	346 500 *c*	0	0	346 500 *c*	10 760 *c*	346 500 *c*	10 760 *c*	Two 15-in. × 55-lb. channels.
Lower chord.	*U o* and *V n*	19.8 *gr*	114 650 *c*	0	0	114 650 *c*	5 790 *c*	114 650 *c*	5 790 *c*	Two 15-in. × 33-lb. channels.
Web members.	*B C*	14.4 *gr*	62 640 *c*	56 990 *c*	28 495 *c*	119 630 *c*	8 300 *c*	135 040 *c*	9 370 *c*	Four 6 × 4 × ⅜-in. angles.
Web members.	*D K*	11.5 *n*	71 820 *t*	56 990 *t*	28 495 *t*	128 810 *t*	11 200 *t*	144 200 *t*	12 540 *t*	Four 6 × 4 × ⅝-in. angles.
Web members.	*F G*	14.4 *gr*	82 500 *c*	56 990 *c*	28 495 *c*	139 490 *c*	9 690 *c*	154 880 *c*	10 750 *c*	Four 6 × 4 × ⅜-in. angles.
Web members.	*H I*	11.5 *n*	68 550 *t*	37 170 *t*	18 585 *t*	105 720 *t*	9 190 *t*	115 750 *t*	10 060 *t*	Four 6 × 4 × ⅝ in. angles.
Web members.	*J K*	14.4 *gr*	80 580 *c*	36 260 *c*	18 130 *c*	116 840 *c*	8 110 *c*	126 640 *c*	8 790 *c*	Four 6 × 4 × ⅜-in. angles.
Web members.	*L M*	8.0 *n*	36 900 *t*	16 460 *t*	8 230 *t*	53 360 *t*	6 670 *t*	57 800 *t*	7 220 *t*	Four 4 × 3 × ⅝-in. angles.
Web members.	*N O*	14.6 *n* / 17.6 *gr*	70 740 *c*	181 280 *t*	18 350 *c*	110 540 *t* / 89 550 *c*	7 580 *t* / 5 090 *c*	217 000 *t* / 89 550 *c*	14 860 *t* / 5 090 *c*	Two 15-in. × 33-lb. channels.

* When one-half the live load is being carried. Itemized loads are given in Table 2.

TABLE 1.—(*Continued.*)

	Member.	Cross-section, in square inches.	Stresses from dead loads, in pounds.	Stresses from live load, in pounds = 87 000.	Stresses from one-half live load, in pounds = 48 500.	Total maximum stresses, in pounds.	Unit stress, in pounds per square inch.	Stresses from Dead Load Plus a Live Load of 110 000 Lb.: Total stresses, in pounds.	Stresses from Dead Load Plus a Live Load of 110 000 Lb.: Unit stress, in pounds per square inch.	Section adopted.
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Web members.	*P Q*	13.6 *n*	39 900 *t*	218 750 *c*	0	39 900 *t*	2 980 *t*	39 900 *t*	2 980 *t*	Two 15-in. × 33-lb. channels.
		19.8 *gr*				178 850 *c*	9 040 *c*	178 850 *c*	9 040 *c*	
	R S	11.5 *n*	157 100 *t*	0	0	157 100 *t*	13 700 *t*	157 100 *t*	13 700 *t*	Four 6 × 4 × 3/8-in. angles.
	T U	21.2 *gr*	221 000 *c*	0	0	221 000 *c*	10 430 *c*	221 000 *c*	10 430 *c*	Four 6 × 4 × 9/16-in. angles.
	A B	8.0 *n*	15 060 *t*	13 050 *t*	....	28 110 *t*	3 500 *t*	31 560 *t*	3 940 *t*	Four 4 × 3 × 3/8-in. angles.
	E F	8.0 *n*	6 625 *t*	0	0	6 625 *t*	830 *t*	6 625 *t*	830 *t*	Four 4 × 3 × 3/8-in. angles.
	I J	8.0 *n*	8 910 *t*	0	0	8 910 *t*	1 110 *t*	8 910 *t*	1 110 *t*	Four 4 × 3 × 3/8-in. angles.
	M N	26.5 *gr*	30 832 *t*	172 467 *c*	9 682 *t*	141 635 *c*	5 340 *c*	239 270 *c*	9 030 *c*	Four 6 × 4 × 9/16-in. angles.
		20.5 *n*				40 514 *t*	1 980 *t*	40 514 *t*	1 980 *t*	One 14 × 5/8-in. plate.
	Q R	28.2 *gr*	166 840 *c*	0		166 840 *c*	5 920 *c*	166 840 *c*	5 920 *c*	Four 6 — 4 × 9/16-in. angles.
		22.0 *n*	24 857 *t* *	0	0	24 857 *t*	1 130 *t*	24 854 *t*	1 130 *t*	One 14 × 1/2-in. plate.
	U V	8.0 *n*	32 606 *t*	0	0	32 606 *t*	4 100 *t*	32 606 *t*	4 100 *t*	Four 4 × 3 × 3/8-in. angles.
	C D	9.9 *gr*	1 500 *c*	0	0	1 500 *c*	150 *c*	1 500 *c*	150 *c*	Four 4 × 3 × 3/8-in. angles.
	G H	9.9 *gr*	1 500 *c*	0	0	1 500 *c*	150 *c*	1 500 *c*	150 *c*	Four 4 × 3 × 3/8 in. angles.
	K L	9.9 *gr*	1 800 *c*	0	0	1 800 *c*	180 *c*	1 800 *c*	180 *c*	Four 4 × 3 × 3/8-in. angles.
	O P	14.4 *gr*	1 800 *c*	0	0	1 800 *c*	125 *c*	1 800 *c*	125 *c*	Four 6 × 4 × 3/8-in. angles.
	S T	9.9 *gr*	8 100 *c*	0	0	8 100 *c*	820 *c*	8 100 *c*	820 *c*	Four 4 × 3 × 3/8-in. angles.
Reactions.	Reaction R_7	....	0	191 700	0	191 700	....	298 000	....	....
	Reaction R_8	(pivot)....	262 908	87 000	235 200	†498 108	....	§498 108	....	....
						‡349 908		‖271 600		
	Reaction R_9	....	191 700	−191 700	−191 700	191 700	....	191 700	....	....

* When one-half the live load is being carried.

† Dead load + one-half live load.

‡ Dead load + live load.

§ Dead load + live load of 48 500 lb.

‖ Dead load + live load of 110 000 lb.

Sinker Castings.—The "sinker castings" are two massive blocks of cast iron (each weighing about 35 000 lb.), which are suspended from the axles of the movable sheaves just mentioned. They are shown on a small scale in Figs. 7 and 11, and in greater detail in Fig. 15. They may also be observed in the photograph, Fig. 6.

TABLE 2.—LOADS CARRIED BY TRUSSES, IN POUNDS.

Loads.		Two trusses.	One truss.
Dead load:			
Structural steel		339 174	169 587
Castings and turning machinery		58 392	29 196
Concrete and windows in machinery room		50 200	25 100
Creosoted timber in footwalks, etc.		10 919	5 460
Hoisting machinery proper	75 450		
Sheaves and brackets	12 100	173 530	86 765
Sinkers and hooks	86 000		
		632 215	316 108
Concrete counterweight		277 000	138 500
Total dead load		909 215	454 608
Live load:			
One stop-log girder in dry		174 000	87 000
Total dead and live load		1 083 215	541 608

The purpose of the "sinker castings" is to increase the weight, thus tending to overcome the frictional resistances when the girders are being lowered in their recesses. This is of special importance in an emergency, when the flow of water through the lock produces lateral forces, increasing the friction. In order to utilize their extra weight, the sinkers, of course, must rest directly on the top of the girders when the latter are being lowered. This was insured (as may be seen in Figs. 11 and 15), by giving an oblong shape to the holes in the castings, through which the axles of the "movable sheaves" pass.

It will also be seen that the structural member from which the hook is suspended passes directly through the sinker casting, and that certain mechanical details, forming a part of the mechanism for detaching the hooks, are fastened to the sinkers.

Attaching of Hooks.—The hooks for supporting the girders when they are being raised or lowered (Fig. 15) are of forged nickel steel and are suspended from pins 5 in. in diameter in a riveted frame within the sinker casting. When carrying their load these hooks engage 6½-in. pins fastened to the inside of the girders. The center lines of the upper and lower pins, when the hooks are attached, are in a vertical plane passing through the center of gravity of the girder, which therefore hangs vertically.

The hook is entered and withdrawn from the girder through a slot 7 in. wide and 2 ft. 3 in. long in the top plate of the girder. As may be seen from Fig. 16, the position of this slot and the shape of the hook are such that the

latter, when lowered, will be deflected sidewise and enter the slot. On being lowered farther inside the girder, the hook swings back to a vertical position, and then, upon being raised, engages the hoisting pin.

Detaching of Hooks.—In order to detach the hooks this process is reversed. They are first lowered slightly to clear the hoisting pin, then deflected sidewise, and, finally, raised through the slot in the top plate of the girder. A separate mechanism is required to give the lateral deflection to the hook.

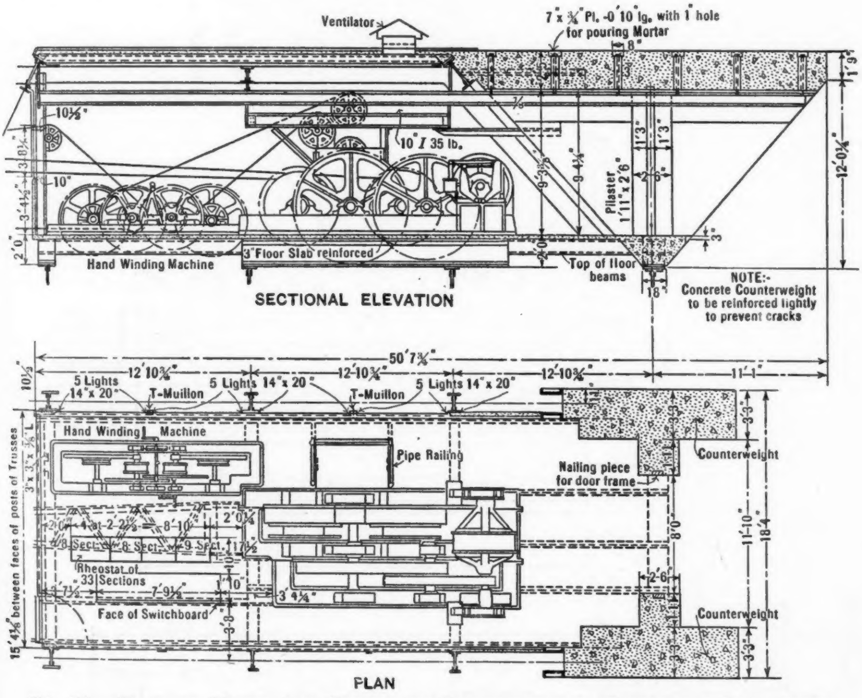

FIG. 14.—PLAN OF MACHINERY HOUSE AND COUNTERWEIGHT, EMERGENCY DAM, INNER HARBOR NAVIGATION CANAL.

Two devices are provided for this purpose. The first of these is the spring shown in detail in Fig. 16. It forms a permanent part of each girder. A stem projecting from the bottom plate near the end of the girder is forced upward when the girder becomes seated, compressing the spring, so that its thrust is transmitted through a bell-crank lever and the hook is pushed sideways. This device, which operates automatically, comes into play after the girders have been deposited in the recesses in the side walls to shut off the flow of water through the lock or to permit its unwatering.

In this case, the ends of the girders come to a solid bearing on the floor, or on the girder next below, and the projecting stem is forced upward. The automatic spring device cannot be used when the girders are placed on their seats in the storage yard, since under these conditions the ends of the girders will project beyond the supports. Similarly, of course, it cannot be used when

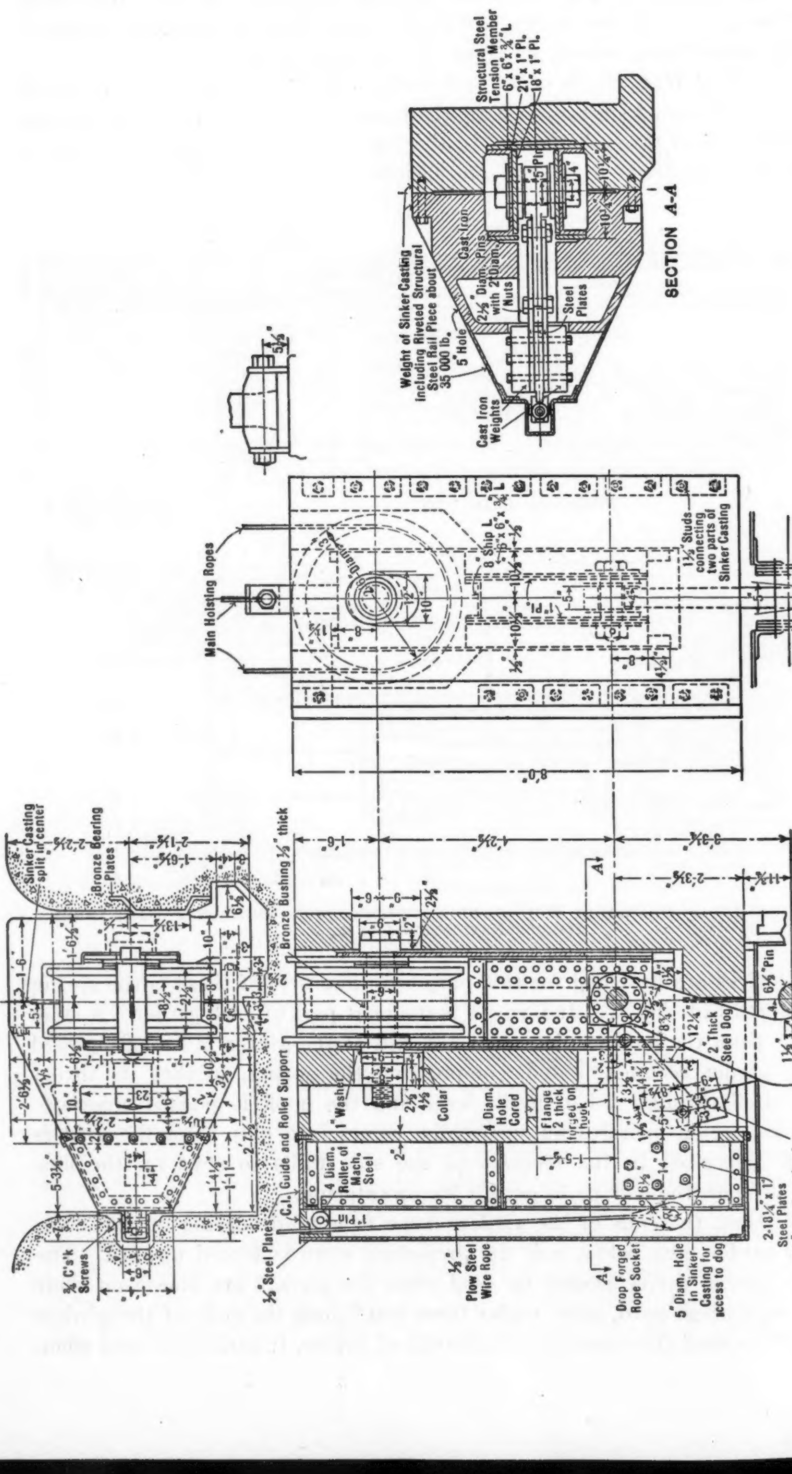

FIG. 15.—DETAILS OF SINKER CASTING, HOOK, AND RELEASING DEVICE, EMERGENCY DAM, INNER HARBOR NAVIGATION CANAL.

the girders are lowered into the water of the lock to be floated to the lower end for use as a coffer-dam. For service in the last two cases, a hand-operated mechanism has been provided. This device is shown in Figs. 11 and 15. Its essential feature is a lever hinged to the vertical flange on the hook and

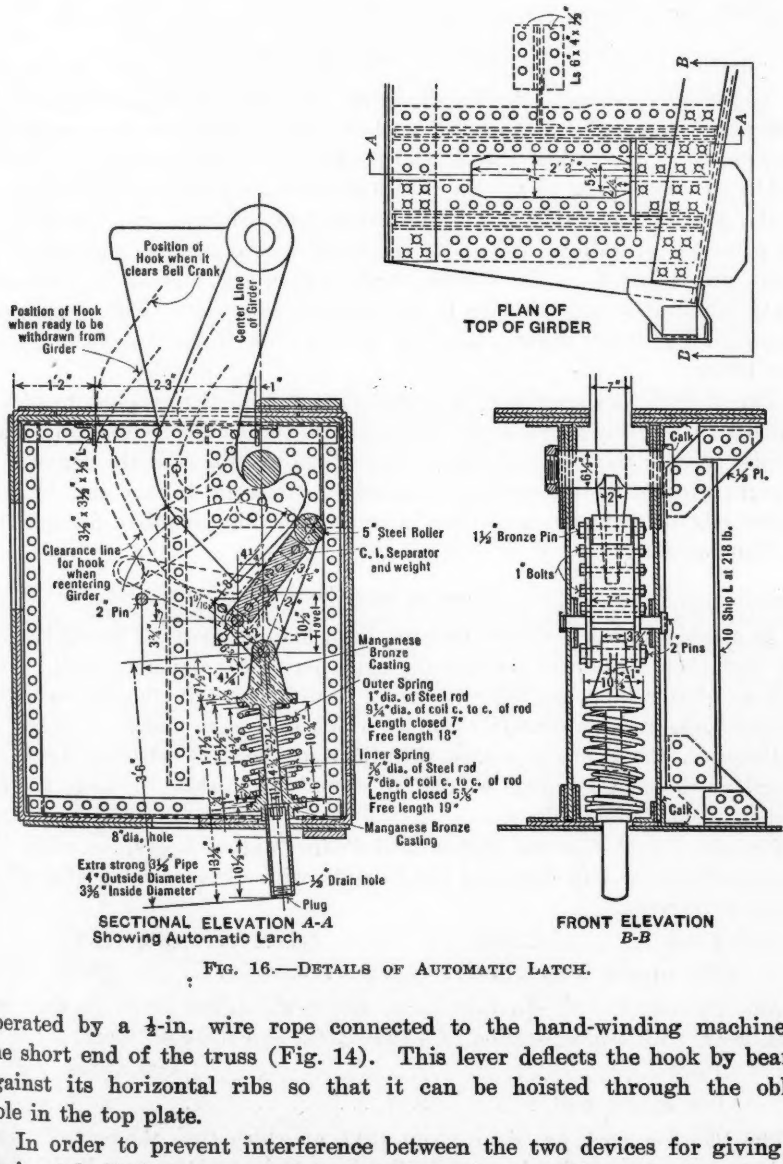

FIG. 16.—DETAILS OF AUTOMATIC LATCH.

operated by a ½-in. wire rope connected to the hand-winding machine on the short end of the truss (Fig. 14). This lever deflects the hook by bearing against its horizontal ribs so that it can be hoisted through the oblong hole in the top plate.

In order to prevent interference between the two devices for giving the horizontal displacement, and to guard against accidents in lowering the girder, the second device is designed so that it will operate only when a 2-in.

block of steel, called "the dog," is in place. When the spring mechanism is relied on to deflect the hook, this dog is released before attaching the hook to the girder, making the hand-operated device inoperative. If, now, in lowering the girder one end becomes wedged in the recess and, at the same time, the ½-in. rope breaks, there will be no danger of the hook becoming detached and allowing the girder to fall.

Operation of the Dam

The motors for revolving the draw-span and for raising and lowering the girders are operated from a small enclosed cab or platform suspended from the truss near the end of the long arm (Fig. 17). This position of the cab enables the attendant to get an unobstructed view of the storage yard and of the girders when they are being swung into position and lowered. All the principal operations are under the control of one man, although a few others are required in the storage yard and on the lock walls. However, this does not mean any addition to the ordinary lock force. Fig. 17 shows the completed draw from above, while Fig. 18 is a view of the dam from the lock floor below.

Experience has shown that it requires 7 min., on the average, to pick up a girder in the storage yard and lower it into the lock recesses, and 12 min. to raise it from the lock and replace it on the back-fill. For the complete set of eight girders these figures correspond to cycles of 1 hour and 1½ hours respectively, or about the same time in each case as that required for operating the Panama dams.

Comparative Costs

As already stated, the New Orleans Dam has the merit of being lower in first cost than structures serving the same purpose previously built. Comparisons of this kind are difficult, owing to differences in the dimensions of various works and the conditions under which they were built.

However, the depth is almost identical in the New Orleans Lock and several of those at Panama, so that it seems worth while to quote the cost of each from the records.

Panama Canal.—In the Gatun and Pedro Miguel Locks, in which the lock chamber is 54.7 ft. deep and 110 ft. wide, with a cross-section of 6 017 sq. ft., the costs were:

Total	$432 475.10
Per square foot	72.00

New Orleans Canal.—In this canal, the lock-chamber is 55 ft. deep and 75 ft. wide, with a cross-section of 4 125 sq. ft., and the costs were:

Total	$353 523.68
Per square foot	85.70

The Panama work was done about 1912, at which time the cost of structural steel was unusually low, while the New Orleans Dam was built under World War conditions, the labor and material at this site being fully twice

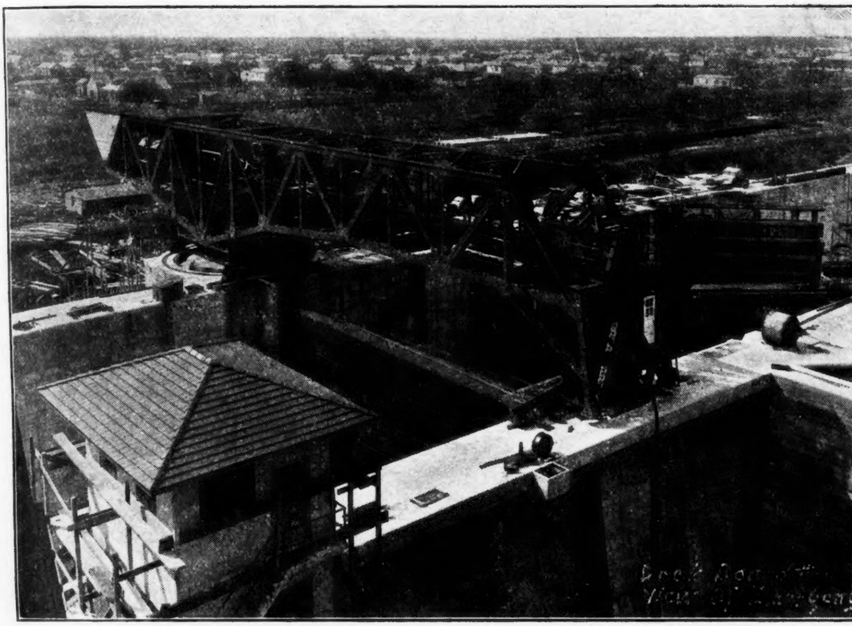

FIG. 17.—VIEW SHOWING EMERGENCY DAM IN TRIAL OPERATION.

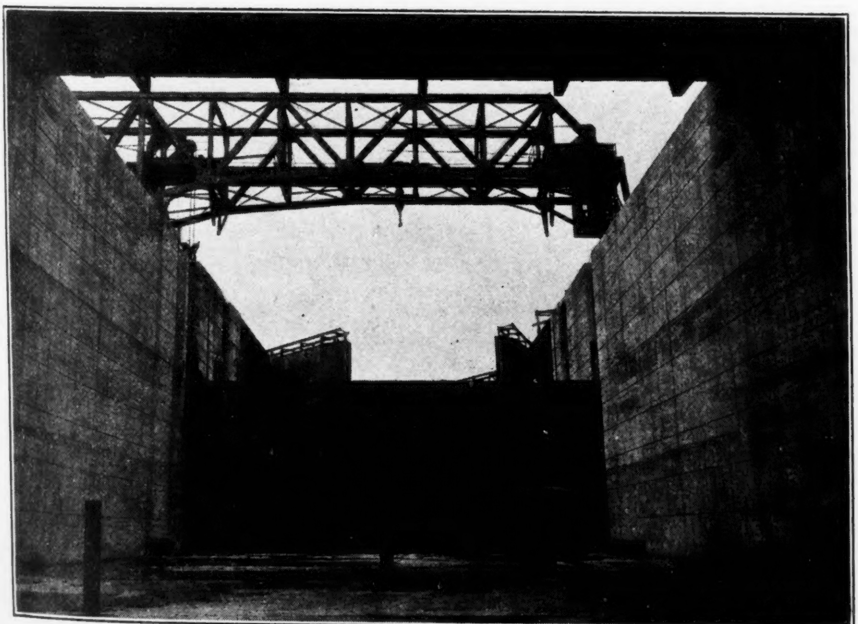

FIG. 18.—VIEW OF EMERGENCY DAM AND SWING SPAN FROM LOCK FLOOR BELOW.

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as high as at Panama in 1912. It seems reasonable, therefore, to ascribe the small increase (about 20%) in the cost per square foot of lock opening at New Orleans, mainly to the more economical type of design.

Contractors and Personnel

The material for the emergency dam, with the exception of the concrete counterweight and some minor parts, was furnished by the following contractors:

Swing span, including turning machinery. Bethlehem Steel Bridge Corporation
Stop-log girders........................ McClintic-Marshall Company
Hoisting machinery.................... Wellman-Seaver-Morgan Company

The structural work was divided between two firms in order to expedite the work which was begun during the World War, its speedy completion being important from a military standpoint.

The Consulting Engineers for the Inner Navigation Canal were George W. Goethals and Company, the writer being retained by the Commission of the Port of New Orleans to prepare the designs for the structural, mechanical, and electrical equipment of the lock. All plans for this part of the work were made in his office. Special credit should be given to F. E. Sterns, M. Am. Soc. C. E., his Principal Assistant.

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UNUSUAL ENGINEERING FEATURES OF AN IMMENSE THEATRE BUILDING

By R. McC. Beanfield,* Assoc. M. Am. Soc. C. E.

Synopsis

Some of the outstanding features of Al Malaikah Temple, at Los Angeles, Calif., are:

- Combination pin and riveted balcony truss, 168 ft. span, weighing 250 tons;
- Steel balcony cantilevers with a 45-ft. over-hang;
- Stage with double asbestos fireproof curtain, probably the largest yet constructed;
- Reinforced concrete stage roof arches supporting steel gridiron, which functions as a tie for the roof arches;
- Seating capacity of 6 550 in orchestra and balcony;
- Reinforced concrete proscenium arch, clear span of 100 ft.;
- Auditorium steel roof trusses, one end on rollers, each weighing 60 tons; span, 192 ft.;
- Banquet hall reinforced concrete roof arches, span, 90 ft.;
- Ventilating system supplying 750 000 cu. ft. of air per min.; and
- Stage switchboard and main chandelier, probably the largest yet constructed.

General Description

The Al Malaikah Temple in Los Angeles, commonly known locally as the Shrine Civic Auditorium, is the new home of the ancient Arabic order, Nobles of the Mystic Shrine. It is located at Jefferson, Royal, and Thirty-second Streets, the site of the old Shrine Auditorium destroyed by fire on January 12, 1920.

The Temple occupies nearly 3 acres (Fig. 1), and may be divided into three units, as follows: The Auditorium, or theatre (Fig. 2), with facilities for housing the various Shrine organizations, such as Patrol, Band, Chanters,

Note.—Written discussion on this paper will be closed in April, 1928.

* Structural and Mech. Engr., Los Angeles, Calif.

etc.; the Banquet Hall (Fig. 4), with a large basement for public exhibition purposes; and a kitchen wing, with facilities for serving large banquets. (See floor plan, Fig. 3.)

The construction throughout is reinforced concrete, except the roof trusses over the Auditorium, the trusses supporting the balcony, and the gridiron over the stage, which are steel.

The Auditorium

The Auditorium Section of the Temple is 200 ft. wide and 294 ft. deep, including the stage. It consists of a reinforced concrete frame with exterior concrete curtain-walls. The roof is constructed with a reinforced concrete joist system, which is supported on structural steel purlin beams connected to the steel roof trusses.

The Auditorium on the south side contains ample dressing-room facilities, including suites for stars. Large dressing rooms for male and female choruses are located in the basement under the stage. The musicians have a rest room and locker facilities. All the dressing rooms are efficiently ventilated and have shower baths.

The Potentate, Divan, Veterans, and various auxiliary organizations of the Shrine, such as the Patrol, Chanters, band, etc., are provided with complete facilities on the second floor on the south side of the Auditorium, which include a kitchen and large dining room.

The Pavilion

The north wall of the Auditorium is equipped with steel rolling fire doors, so that it can be shut off entirely from the Banquet Hall when desired. The large Banquet Hall, or Pavilion, contains 101 000 sq. ft. of floor space, including a large basement used for exhibition purposes; also, ground and second floors with a surrounding cantilevered balcony. There are two adjoining kitchens and serving rooms of capacity large enough to serve 10 000 people. The reinforced concrete roof of the Banquet Hall (Fig. 4), is supported on a series of beautiful reinforced concrete Saracenic arches, having a clear span of 90 ft.

The entire ground floor of the Pavilion is finished with white maple flooring, laid on a patented floor filler, called "Nailcrete". The Pavilion is used for art exhibits, civic expositions, and large balls. It has a clear ceiling height of 17 ft. in the basement, where machinery or automobile displays can be installed. Automobile ramps are provided from the Thirty-second Street front to this level. The basement is ventilated by a forced circulating system, the foul air being exhausted at the floor level in order to accommodate automobile exhibits.

The Stage

From a theatrical viewpoint, the Auditorium is one of the largest constructed to date (Fig. 2). Its stage, 72 ft. deep and 192 ft. wide, one of the largest in the world, was designed to give ample room for the elaborate rituals

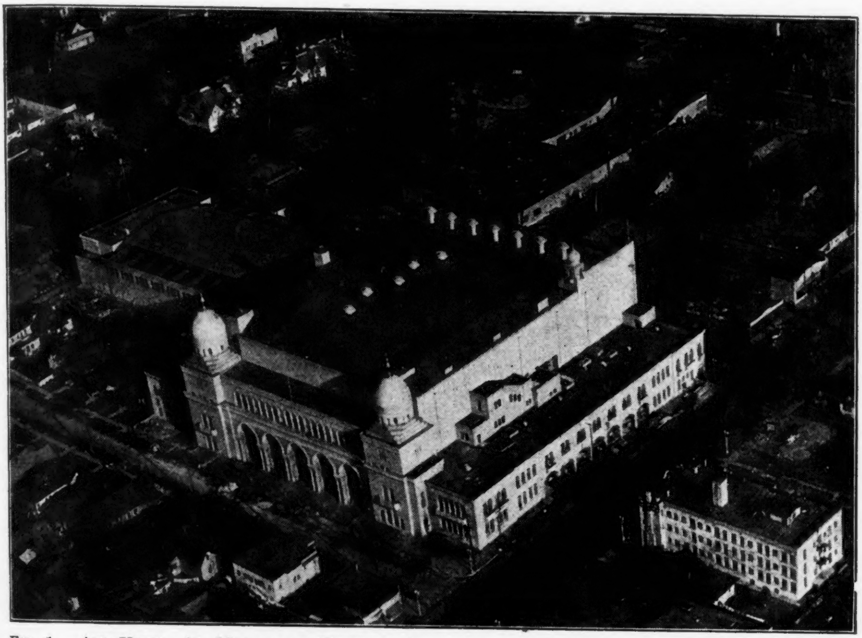

FIG. 1.—AIR VIEW, AL MALAIKAH TEMPLE, AUDITORIUM IN FRONT, BANQUET HALL AT LEFT.

FIG. 2.—VIEW OF INTERIOR OF THEATRE, SHOWING ABSENCE OF COLUMNS. CHANDELIER IS 22 FEET IN DIAMETER, GIVING IDEA OF SIZE.

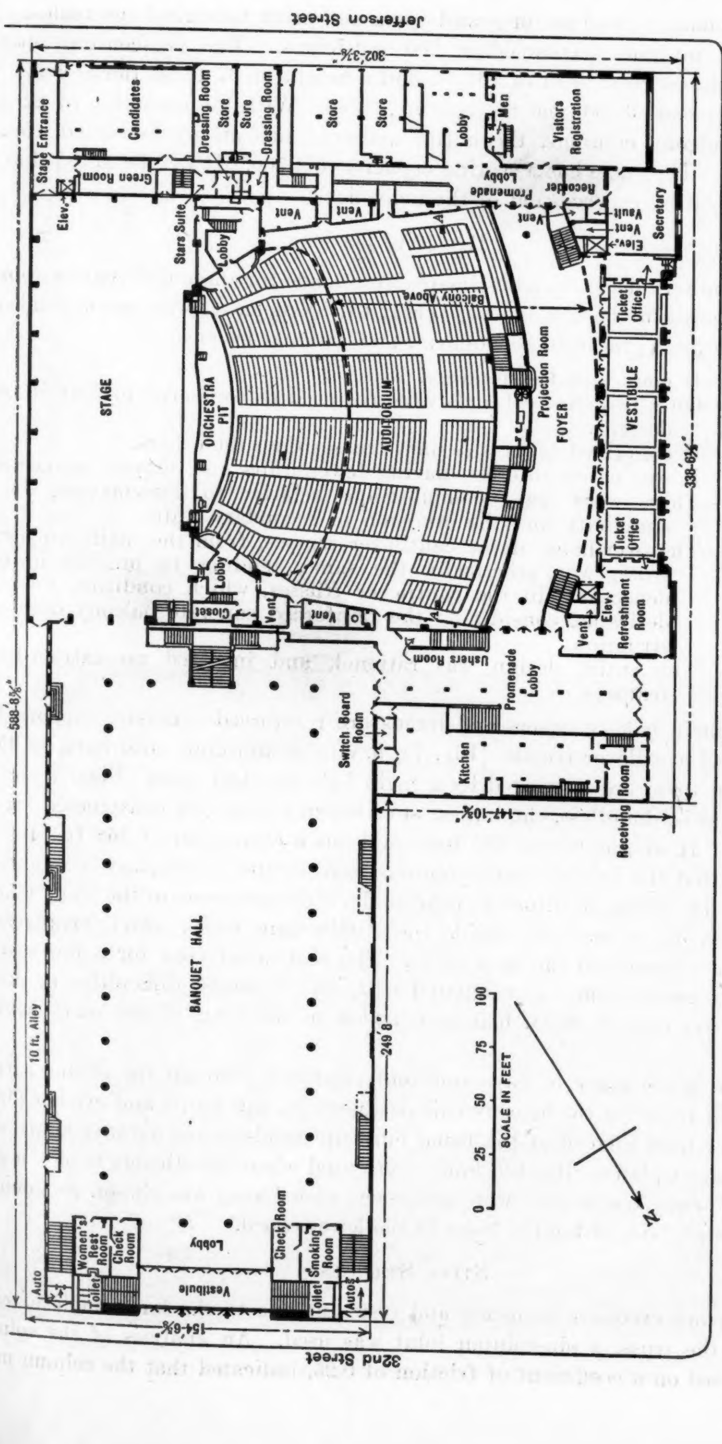

FIG. 3.—FIRST FLOOR PLAN, AL MALAIKAH TEMPLE.

of Shrinedom, as well as for grand opera and great theatrical spectacles. The orchestra pit can accommodate 200 musicians. The proscenium opening (Fig. 5) has a clear span of 100 ft. and a height of 37 ft. at the crown. The main floor seats 3 200, and the balcony, 3 350. With the exception of 22 seats behind balcony columns, the entire audience has an unobstructed view of the stage. The stage has a seating capacity of 1 700. It will be noted that the balcony seating exceeds that of the main floor.

Balcony Trusses

Preliminary designs and estimates were made for three different systems of truss framing under the balcony (Fig. 6). Scheme *C* was recommended as being the most practical, economical, and rational because:

1.—It contained less tonnage.

2.—Only two steel columns were required as compared to four in other designs.

3.—It contained fewer auxiliary trusses than the others.

4.—It was easier to erect, having fewer units and simpler connections.

5.—There were fewer complications relative to deformation, due to simplicity and the omission of secondary units.

6.—The over-hang of the cantilever trusses, from the main supporting truss, was practically the same, tending to provide uniform deflections in the cantilever trusses, which condition was very desirable, considering the reinforced concrete balcony deck construction.

7.—The entire design was rational, and involved no extraordinary features.

The main balcony risers and treads are reinforced concrete, supported by eight steel cantilever trusses (Fig. 7), having a minimum over-hang of 45 ft. which, in turn, are supported by a main balcony steel truss (Figs. 7 and 8). This truss is, doubtless, the largest steel balcony truss yet constructed for the purpose. It weighs nearly 250 tons, and has a clear span of 168 ft. It is of interest that the balcony truss columns were located to displace a single seat on each side of the Auditorium nearest the aisles adjacent to the walls (shown at *a*, Fig. 3), instead of outside the Auditorium walls, which arrangement would have increased the span 20 ft. The obstructed view for a few seats is relatively unimportant as compared with the increased difficulties in design and greater cost if 20 ft. had been added to the span of the main balcony truss.

As it is necessary to have sufficient clearance through the center part of this main truss for the balcony exit passageways, pin joints and eye-bar diagonals were used instead of the usual built-up members and riveted joints with larger gusset-plates. Riveted joints were used where practicable to obtain general stiffness. Where pins were necessary, nickel steel was chosen particularly to reduce the size of the pin-holes in the lower chord.

Steel Stresses

To avoid excessive moments and restraint due to the large angle of rotation in the truss, a pin-column joint was used. An analysis of the column joint based on a coefficient of friction of 0.25, indicated that the column must

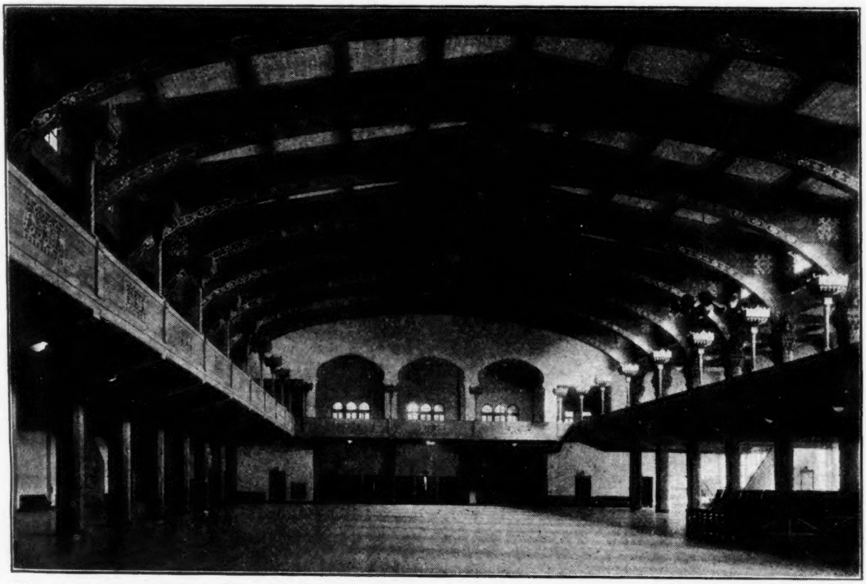

FIG. 4.—CONTINUOUS ROOF GIRDERS IN BANQUET HALL, SPAN 90 FEET. ANCHOR ARMS PROJECTED ABOVE LOWER ROOF LEVEL. DECORATIONS APPLIED DIRECTLY ON CONCRETE.

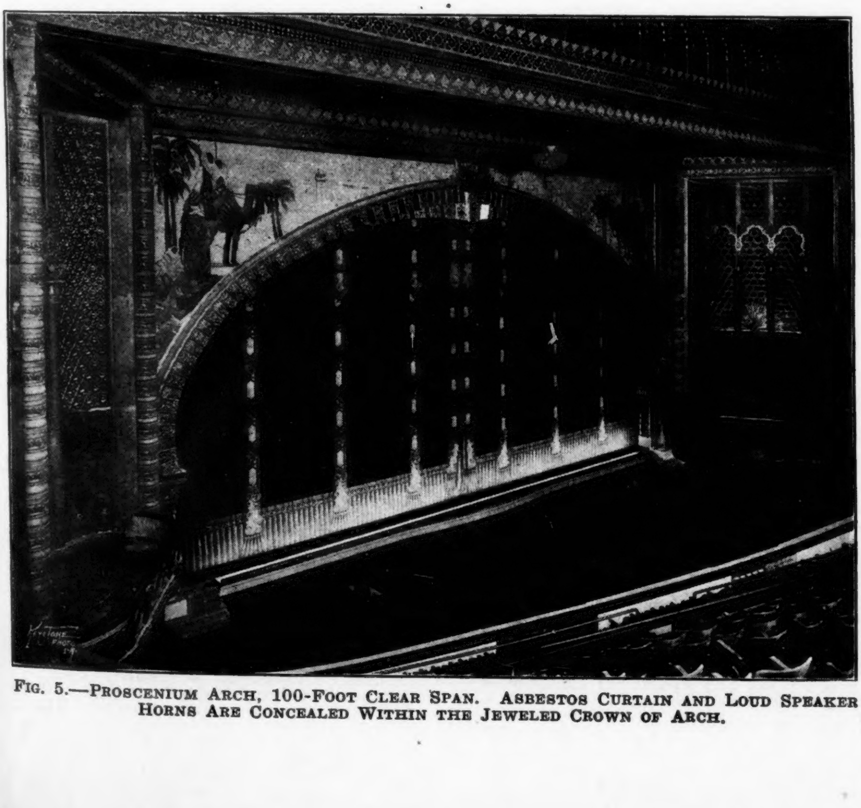

FIG. 5.—PROSCENIUM ARCH, 100-FOOT CLEAR SPAN. ASBESTOS CURTAIN AND LOUD SPEAKER HORNS ARE CONCEALED WITHIN THE JEWELED CROWN OF ARCH.

resist moment due to pin friction before the pin could rotate. Few data are available relative to the friction of pins in steel trusses. This coefficient (taken from Professor Föppl) is, at best, only an approximation for this condition.

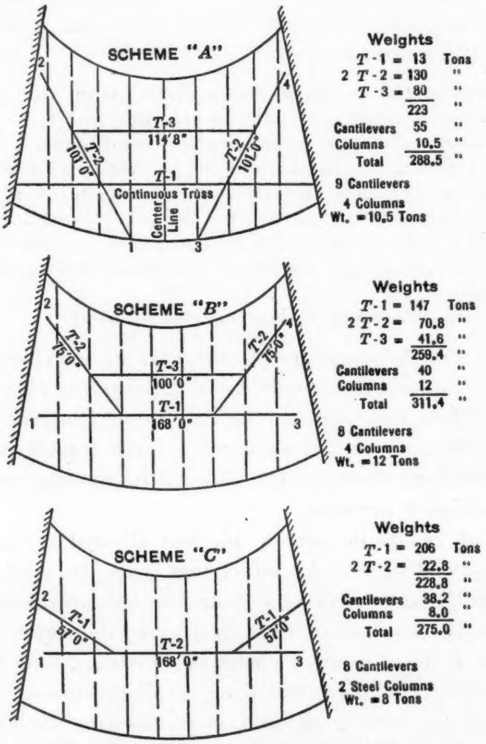

FIG. 6.—STUDIES FOR STEEL TRUSSES UNDER BALCONY, SHRINE AUDITORIUM.

Except for the stresses, the design of the balcony truss was based on the report of the Society's Special Committee on Specifications for Bridge Design and Construction.* The stresses recommended by the American Institute of Steel Construction were adopted. The stresses used for nickel-steel pins were 36 000 lb. per sq. in. in the extreme fiber for bending, 30 000 lb. per sq. in. for bearing, and 20 000 lb. per sq. in. in shear.

SECONDARY STRESSES

Due to the unprecedented size and unusual design of the main balcony truss, *T*-2, with its combination pin-and-riveted connections, a careful analysis of the secondary stresses was made. The computed restraining moments at the pin joints were rather uncertain as the coefficient of friction of the nickel-steel pins was an unknown factor and had to be assumed.

* Final Report of the Special Committee on Specifications for Design and Construction of Steel Railway Bridge Superstructure, *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 471.

An adaptation of Mohr's semi-graphical analytical system, by Mr. G. A. Maney,* was selected by the writer for calculating the secondary stresses. Maney's arrangements of Mohr's methods are much simpler, more concise, and involve less time than other commonly accepted methods.

Only approximate results to determine probable localized excessive stress conditions were expected, on account of the modifying effects of various conditions, such as the following:

(1) Change in the primary stress distribution due to changes in the section of members (particularly at joints), to gussets and splice members, and also to fabrication conditions.
(2) Effects of changes in moments of inertia in members between joints.
(3) Variation between the theoretical length of a member between axial intersection points and the actual length between gussets.
(4) Elastic deformation of the gusset-plates, initial slip, and distortion of the rivets, and degree of frictional resistance between riveted plates.
(5) Unknown frictional resistance of pin joints.

In computing the deformations in the various members, the gross areas were used, no allowance being made for details or for changes in section at the connections. Mohr's successive approximations (to the third substitution) were utilized, which gave results accurate enough for all practical purposes. All computed secondary stresses were checked by another method of calculations to results within 5 per cent.

A maximum of 24 000 lb. per sq. in. was allowed for combined stresses, including secondary stresses. An effort was made to modify the secondary stresses due to dead load conditions by lengthening and shortening the various truss members in the amount of their respective deformations, which necessitated the use of hydraulic jacks, steamboat ratchets, and tapered drift-pins to force the members into their riveted or pin connections. As the dead load was about 72% of the total load, and as it was improbable that the entire area of the balcony would ever be loaded with the computed live load of 75 lb. per sq. ft., the seats being fixed, the writer deemed it advisable to try, by proper erection methods, to eliminate in some degree the secondary stresses due only to dead load conditions. While considerable force had to be used to spring the various members into place, little difficulty was experienced.

Due to the comparatively small deformations of the truss members under dead load conditions (Fig. 9), and the impracticability of obtaining such extreme accuracy in measurements in the structural shop work, secondary stresses probably were not eliminated to the desired extent, as subsequent strain-gauge measurements have indicated.

Some difficulty was encountered with hand-reaming. Due to large holes (1⅛ in.) and thick built-up members, it was difficult for the reamer crews to keep the reamers steady, there being a tendency to wobble the reamer, which resulted in a few over-sized holes.

* "Secondary Stresses and Other Problems in Rigid Frame," *Bulletin*, Univ. of Minnesota, March, 1925.

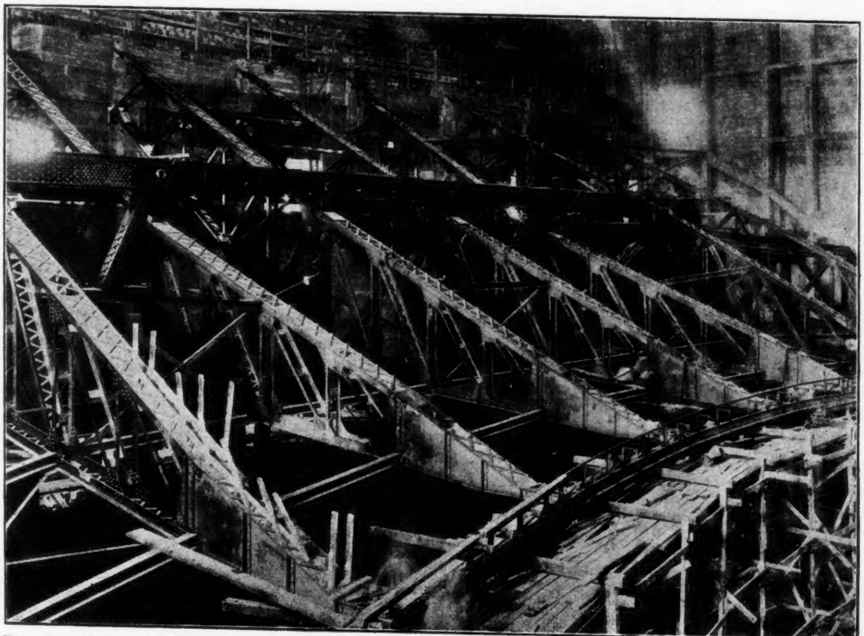

FIG. 7.—BALCONY TRUSSES CANTILEVERED FROM MAIN TRUSS, T-2. JACK TRUSS, T-1, AT RIGHT, SUPPORTS TWO CANTILEVERS.

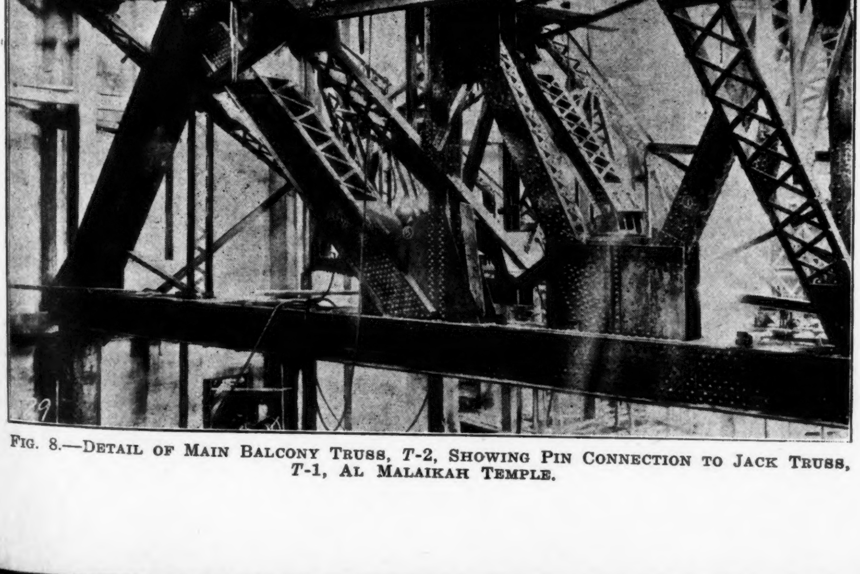

FIG. 8.—DETAIL OF MAIN BALCONY TRUSS, T-2, SHOWING PIN CONNECTION TO JACK TRUSS, T-1, AL MALAIKAH TEMPLE.

Balcony Truss Erection

Some doubt had been expressed regarding the ability and facilities for manufacturing the large truss near-by. Its successful fabrication and erection by a Los Angeles contractor (The Llewellyn Iron Works) was an achievement that created considerable local pride.

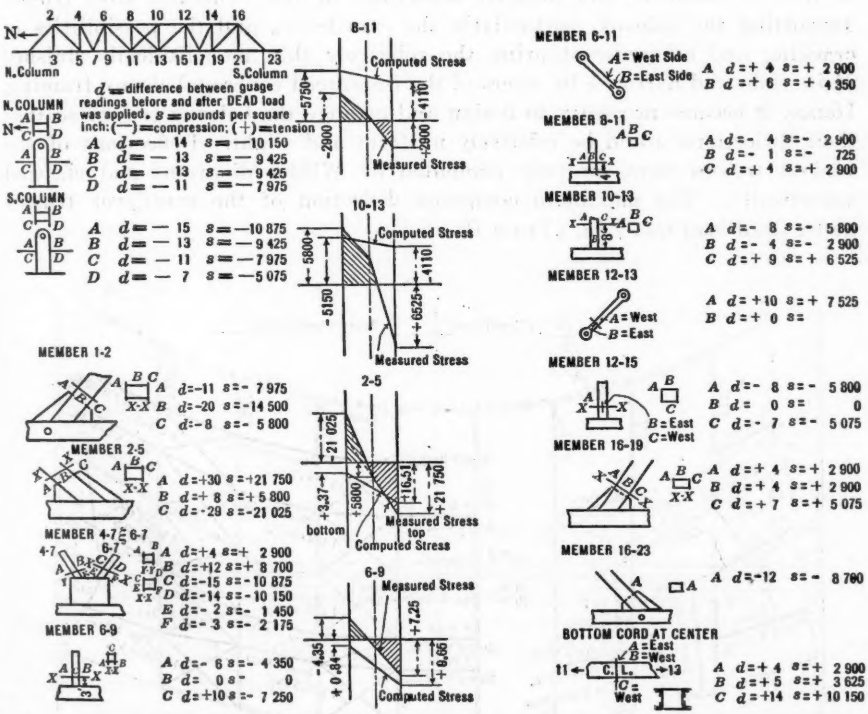

Fig. 9.—Strain-Gauge Investigations, Main Balcony Truss, T-2, Al Malaikah Temple.

This balcony truss was delivered to the job in sections, the largest (center lower chord section) weighing 47 tons and being 80 ft. long. The truss was erected on a steel substructure consisting of the same members as were used in the falsework for the roof trusses. The lower chord sections were first set on the camber screw-jacks. The web members were next placed, followed by the top chord sections.

No unusual difficulty was experienced in forcing the various members into their connections, and the pins were driven home without any undue effort. Prior to riveting, at least 30% of the holes in the connections were bolted tightly. No trouble was encountered in driving the 1-in. rivets (some with a maximum grip of 5¾ in.) by ordinary pneumatic hammers, with 120-lb. air pressure. Comparatively few rivets were rejected. The 1-in. rivets known as "swell-neck" rivets, were tapered a full 1/16 in. for 1 in. along the shank, starting at the head.

The taper on large rivets helped materially to fill the holes completely, particularly in over-reamed holes. In general, they resulted in fewer loose rivets, especially where the rivet grip was large.

The general layout of the balcony framing is shown in Fig. 10. The jack trusses, T-1, tend to equalize the overhang of the cantilever trusses, which project a maximum of 45 ft. beyond the main balcony truss. It was desirable to prevent excessive and unequal deflections of the structural steel trusses supporting the balcony, particularly the cantilevers, and the possibilities of cracking and otherwise shearing the relatively thin monolithically finished $2\frac{1}{2}$-in. slabs and narrow 5-in. risers of the reinforced concrete balcony framing. Hence, it became necessary to design and arrange the balcony trusses so that their deflections would be relatively uniform and slight. Deflections of the various trusses were carefully computed by Williot diagrams and checked analytically. The maximum computed deflection of the cantilever trusses under dead load was $\frac{3}{4}$ in. (Truss B).

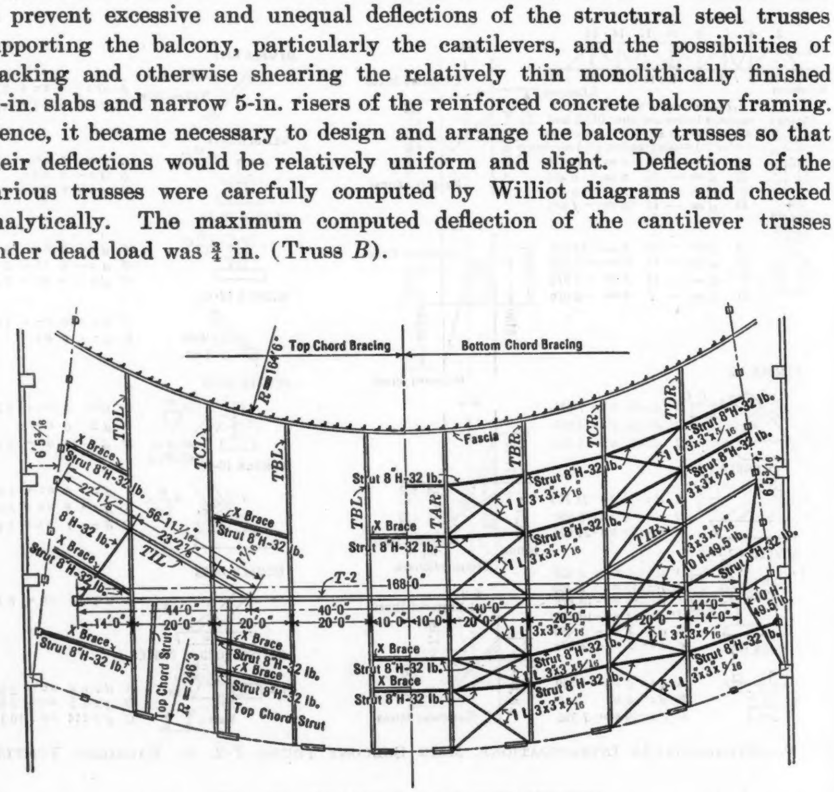

FIG. 10.—BRACING FOR BALCONY TRUSSES.

The cantilever balcony trusses were cambered for dead load by offsetting the rivet holes at the fulcrum connections in the gusset-plates on the main truss or the jack trusses, as the case may be, the top holes being set back and the bottom holes brought forward. The rivet holes in the gusset-plates at the fulcrum connecting the cantilever trusses to the main truss or jack trusses were drilled and reamed in the field after the cantilever trusses had been supported in their proper cambered positions by cables with turnbuckles secured to the roof structure. In adjusting the cantilever trusses to their proper camber the milled joint at the heel or fulcrum in the bottom chord was displaced so that the abutting faces in the joint were not in proper contact. This was corrected in a very satisfactory manner by filling the gap with electrically welded metal.

Electric Welding

Electric welding was found to be an excellent method for obtaining full metal-to-metal contact where milling of inclined surfaces cannot be made uniformly tight. The joint should be properly scarfed to allow the metal to be fluxed or built in to the inner side of the joint. Electric welding does not overheat, and possibly injure, the adjoining metal as will often result from the use of the oxy-acetylene torch in such operations.

Cutting Out Rivets

Where rivets had to be removed the oxy-acetylene torch was used to good advantage to burn off the button of the rivet. This operation, however, should be performed only by a mechanic experienced in using the torch, otherwise the metal in the joint may be injured. The cutting flame should be applied to the side of the rivet head at right angles to, rather than parallel with, the shank of the rivet. It is not always necessary to burn off all the metal in the rivet head, nor should the cutting flame enter the rivet hole.

Cutting out rivets by the oxy-acetylene torch is preferable to the usual method of knocking off the head by heavy sledges and incidentally loosening or jarring other rivets in the same group as well as possibly destroying the frictional resistance between plates in the joint.

Extensometer Readings

Prior to enclosing the balcony steel with a 2-in. thickness of cement mortar as fireproofing, a series of extensometer measurements was made on various members to ascertain to what extent the secondary stresses, due to dead load conditions, were relieved or modified, and to obtain a check on some of the primary stresses. All measurements were made with an 8-in. Berry (lever-type) gauge. Comparative measurements were made on a standard bar for temperature corrections.

The gauge holes were made with a breast drill, using a $\frac{3}{32}$-in. bit; the surface was reamed with an $\frac{1}{8}$-in. bit. Extreme care was necessary to obtain reliable and accurate readings. The average of three measurements was used. A slight difference in pressure on the points of the gauge when applying the extensometer produced considerable variation in the readings. The points on the gauge seemed to have too great a taper, being about 60 degrees. Points with sharper tapers, about 30°, and smaller gauge holes, properly reamed, doubtless, would give more accurate readings.

Readings to the number of 360 were taken on the balcony truss. Due to obstructions and inaccessibility, it was impossible to obtain complete extensometer readings on all truss members, and at many desirable points, to obtain the maximum secondary stresses. However, sufficient strain-gauge measurements were taken to demonstrate, in a practical way, what members were under maximum stresses. From the results obtained it appears that the stresses in the members varied considerably over the cross-sections, and that the maximum measured stress was well within safe limits.

The measurements indicated that the results of the field methods to modify the computed secondary stresses were rather uncertain, possibly due to relatively small deformations and inaccuracy in shopwork, and that the calculated secondary stresses did not fully agree with the measured stresses (Fig. 11). Doubtless this was due to the several modifying effects already mentioned, which are not subject to analytical calculations.

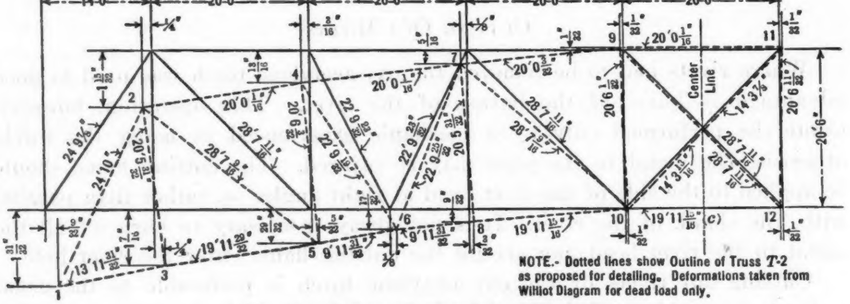

FIG. 11.—STRAIN-GAUGE INVESTIGATIONS.

CALCULATED *Versus* ACTUAL STRESSES

Although there was a lack of agreement between the computed and measured secondary stresses in this particular structure, the efforts spent to calculate the secondary stresses were not entirely wasted, because the analysis indicated to the designer which members in the structure were expected to develop maximum secondary stresses. The primary stresses were reduced, in accordance with the judgment of the designer, in those members in which excessive secondary stresses were indicated. The most highly stressed members according to the secondary stress calculations were found to have the maximum measured stresses.

It is interesting to note that the strain-gauge readings on the main steel columns supporting the balcony truss indicated a small moment with a maximum stress of 10 875 lb. per sq. in., which was safe, being about 30% greater than the computed stress. These columns were designed with a very low unit stress, 8 500 lb. per sq. in., to allow for increased stresses due to bending and other causes, which could not be computed accurately. Each of these columns is braced near its top by three 10-in. H-struts to the reinforced concrete wall structure. Note the variation in stress distribution on the section shown in Fig. 11.

BALCONY DESIGN

Truss *T*-1 is attached at one end to the main balcony truss by a pin connection in a large steel casting resting on a built-up stool between the web members of Truss *T*-2 (Fig. 8). The other end is supported on a reinforced concrete wall column. The chords of Truss *T*-1 consist of two 15-in. channels, with 8-in. Bethlehem H web members. The bottom chord

in the end panel section of the balcony truss, T-1, was composed of two 8-in. standard eye-bars to avoid clearance difficulties. The pin joint connecting Truss T-1 to the main balcony truss, T-2, eliminated any possibility of excessive restraining moments due to the varying deformations of the connected trusses.

There are eight cantilever trusses (Fig. 10), four of which are directly supported and connected on the main balcony truss, T-2; the other four are supported and connected at their fulcrum points to the jack trusses, T-1. The center four cantilever trusses are riveted at their fulcrum supports directly to the vertical member of the T-2 main balcony truss, which is common to both trusses (Fig. 7).

The load on the main balcony truss columns is distributed by a 52 by 52 by 4-in., rolled steel slab to the reinforced column below. It was found to be more economical to use a reinforced concrete pier from the foundations to the first floor level than to extend the steel columns to the footings.

The structural detailing of the balcony framing, which was done by the Llewellyn Iron Works, of Los Angeles, and checked under the writer's direction, involved some very intricate problems, especially with reference to clearances. That only two minor errors, due to changes in plans, occurred in the field erection is an accomplishment worthy of mention.

In order to stiffen the ends of the cantilever trusses and reduce their deflections to a minimum, a double web plate girder section was used, as shown in Fig. 7. During erection, the plate girder sections were given a camber by offsetting the rivet holes in the bottom chord in the joint connecting them to the open web section of the truss. The chords were built of 8 by 3-in. angles, latticed and placed 1 ft. 3 in., back to back, which width added to the lateral stiffness. The anchor, or wall, ends of the cantilever trusses were secured to anchor-bolts on top of reinforced columns.

Special attention was given to the design of the general bracing throughout the balcony framing. The cantilever trusses were sway-braced in horizontal and vertical planes by angle cross-bracing and struts in the form of bents, as indicated in Fig. 10. The cantilevers and jack trusses assisted materially in bracing the main balcony truss in addition to the four horizontal boxed latticed struts which braced the upper chord to the rear wall (Fig. 8). The four exits passing through the main truss were supported by hanger rods anchored above in the concrete deck of the balcony.

To avoid any possibility of the concrete balcony cracking along the Auditorium walls, due to deflections of the supporting trusses, a construction joint was placed in the concrete deck, midway between the Auditorium wall and the adjacent cantilever trusses. Although the balcony has been loaded to capacity several times, no cracks in the concrete decking have been observed to date.

The local building ordinance required a full live load test of the balcony, which was accomplished by using metal plaster barrels filled with water. After the test the water was siphoned from the barrels.

Unusual Stage Construction Features

The stage roof arches, eight in number, are spaced 22 ft. on centers. One end of each of the center arches is supported by the proscenium arch wall. The arches were designed as partly fixed (Fig. 12). Turnbuckles were placed in the bars near the west wall to develop equal stresses in the two channels that form the horizontal tie and to produce a small amount of initial stress in the tie prior to stripping the forms. The gridiron floor proper consisted of flats 4-in. by $\frac{3}{8}$-in., spaced 7 in. on centers, which construction resulted in considerable saving as compared with the usual channel sections. The gridiron floor was designed for a live load of 100 lb. per sq. ft., which has proved ample for stage operating conditions.

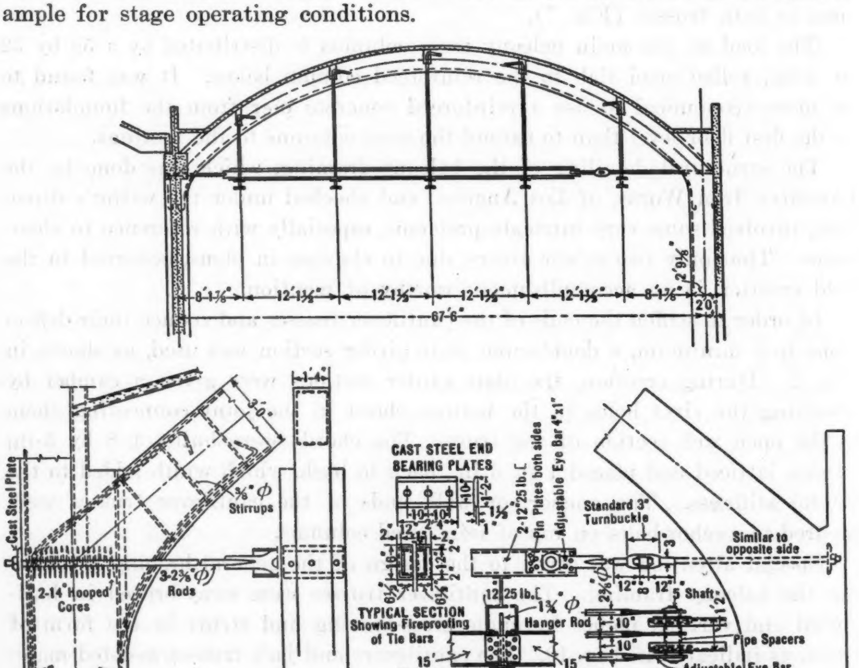

Fig. 12.—Stage Roof Arches.

The stage roof arches were poured in one continuous operation, commencing from each springing line and working simultaneously and symmetrically toward the crown. The arches and the beam system were poured up to the under side of the roof slab. By casting the arched roof slab with concrete having a slump of $2\frac{1}{2}$ in., cover boards were found to be unnecessary. The roof arches were cast with a concrete having the proportions of 1 part cement to 4.5 parts aggregate.

The proscenium arch (Fig. 13), has a clear span of 100 ft. This arch was computed as a girder with partly restrained ends. Particular attention was given to diagonal tension stresses, which were resisted by diagonal bars

(Fig. 13). In order to increase the lateral stiffness, pilasters were placed in the web under each roof arch similar to the stiffeners in plate girder construction. A flanged head, or tee, in the top of the proscenium girder also adds to its lateral stiffness.

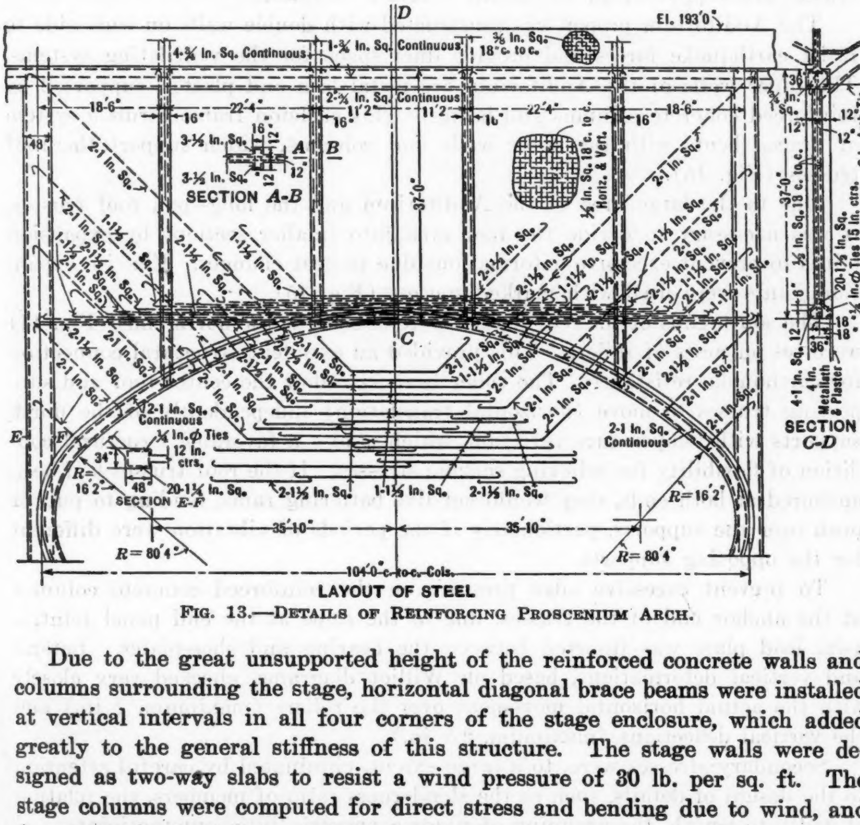

FIG. 13.—DETAILS OF REINFORCING PROSCENIUM ARCH.

Due to the great unsupported height of the reinforced concrete walls and columns surrounding the stage, horizontal diagonal brace beams were installed at vertical intervals in all four corners of the stage enclosure, which added greatly to the general stiffness of this structure. The stage walls were designed as two-way slabs to resist a wind pressure of 30 lb. per sq. ft. The stage columns were computed for direct stress and bending due to wind, and for partly fixed conditions of the roof arches.

The local building ordinance required that the following column reduction load formula be used, when the unsupported length exceeds fifteen times the least diameter:

$$P' = P\left(1.6 - \frac{L}{25\,D}\right)$$

in which,

P' = permissible load.
P = permissible load due to ordinary column formula.
L = unsupported length.
D = least dimension of core.

The stage floor was designed for a live load of 250 lb. per sq. ft. The framing consists of standard steel I-beams supporting a wood-joist system. All steel connections were bolted so that any particular section of the floor could be removed readily without affecting the adjacent parts.

Auditorium Roof Trusses

The structural steel roof trusses over the Auditorium (Fig. 14), each weighing 60 tons, have a clear span of 192 ft. It will be noted that these trusses are supported on reinforced concrete columns.

The Auditorium proper was constructed with double walls on each side to resist earthquake forces and provide duct space for the ventilating systems. The inner walls of the Auditorium are metal lath and plaster, supported by reinforced concrete columns and beams. This skeleton frame forms a system of braced bents with the outer walls and columns, which support the roof trusses (Fig. 15).

Due to the large area of the Auditorium and the long-span roof trusses, it was necessary to divide the roof area into smaller sections by expansion joints to prevent excessive deformations due to heat changes. Elastite expansion joints were used with marked success (Fig. 15).

The south ends of the roof trusses were anchored; the north ends (Fig. 14) were set on nests of rollers. This provided an excellent structural connection for earthquake resistance. The roller bearings allow the center roof and supporting trusses to move (horizontal translation) independently of the north supports with temperature variation, which is also a favorable structural condition of flexibility for relieving seismic stresses. If the roof trusses had been anchored at both ends, they would act like battering rams, tending to pull or push over the supports, particularly if the periods of vibration were different for the opposing supports.

To prevent excessive edge pressure on the reinforced concrete columns at the anchor end of the trusses, due to the slope at the end panel joint, a $\frac{1}{8}$-in. lead plate was inserted between the bearing and shoe-plates. Lateral and vertical deformations, based on Williot diagrams, checked very closely with the actual horizontal movement over the rollers (maximum, $\frac{3}{8}$ in.) and the vertical deflections (maximum, $2\frac{7}{16}$ in.).

Secondary stresses were, to a large extent, eliminated by careful attention to the design of details, such as the slenderness ratio of members, the relation of width to length, the grouping of rivets concentrically about the joints, and the agreement of the neutral axis with the gravity axis, etc. The members were liberally proportioned for the primary stresses to avoid the danger of being overloaded by the secondary stresses. The results of the calculations indicated that the combined primary and secondary stresses did not exceed 21 000 lb. per sq. in. Because of the relative low secondary stresses, it was not deemed necessary to attempt to eliminate them by shop or erection methods.

The steel roof trusses were erected in pairs from a movable substructure (Fig. 14), consisting of braced steel bents bolted together and resting on rollers over timber cribbing. After all the roof trusses were erected, the movable steel substructure was dismantled and its members were used in the temporary supports for erecting the main balcony trusses.

The roof trusses were first assembled in the shop to the proper cambered positions, reamed, and match-marked. The trusses were erected by sections. The bottom chord was assembled first on the camber blocks of the movable

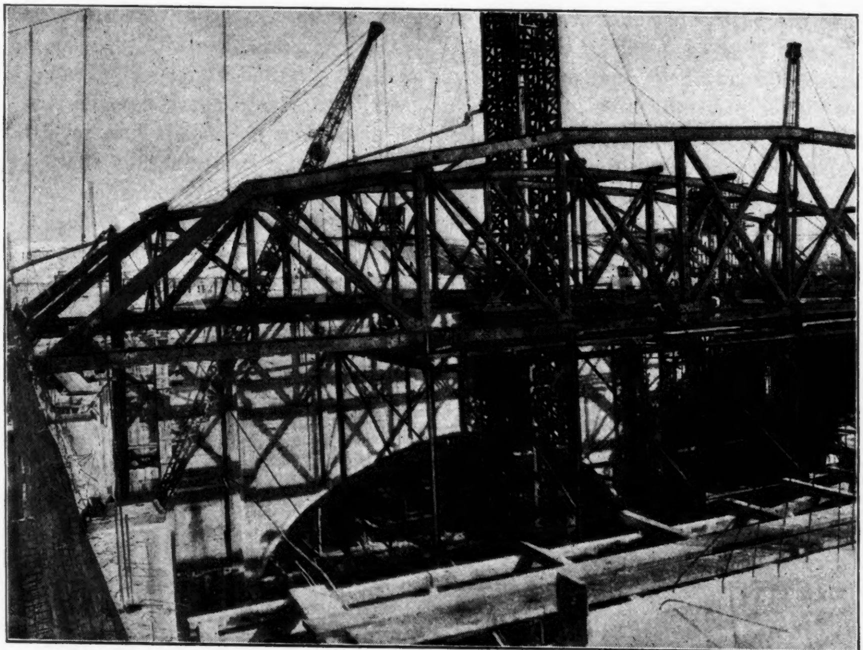

FIG. 14.—ROOF TRUSSES, ROLLER END SHOWING MOVABLE STEEL FALSEWORK, AL MALAIKAH TEMPLE.

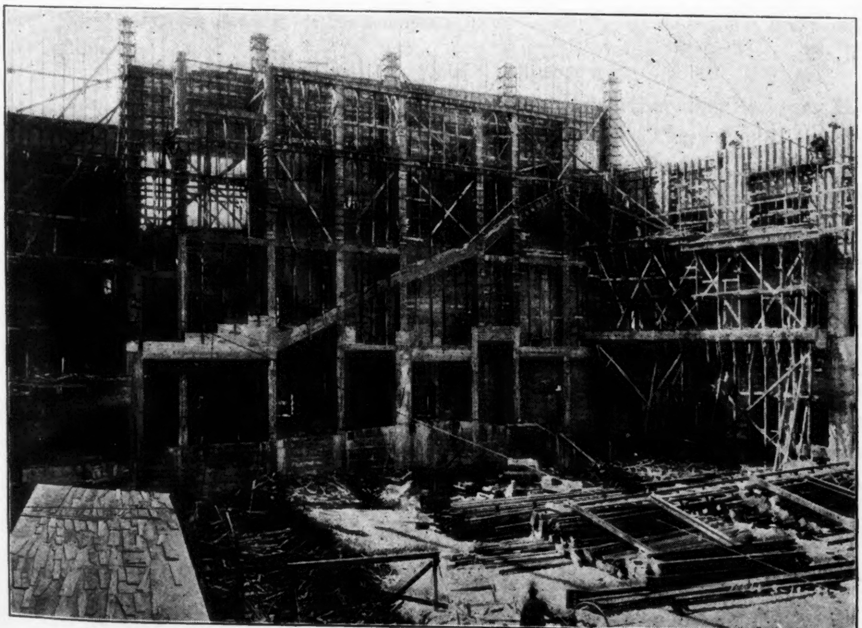

FIG. 15.—VIEW OF INNER WALL FRAMING AND SOLID EXTERIOR SIDE-WALL, AL MALAIKAH TEMPLE.

substructure, then the web members were installed, followed by the top chord sections. All members were bolted tightly with at least 30% of the rivet holes in the joints bolted. A great saving in the roof steel (about 14 tons) was effected by using a cantilevered purlin system, with alternate double cantilever and suspended members.

The computations relative to cantilever purlins are based on full uniform load conditions, in which case the most economical cantilever projection is 0.1465 of the distance between the trusses. For partial loading, the maximum moment due to uniform loads would be increased 20 per cent.

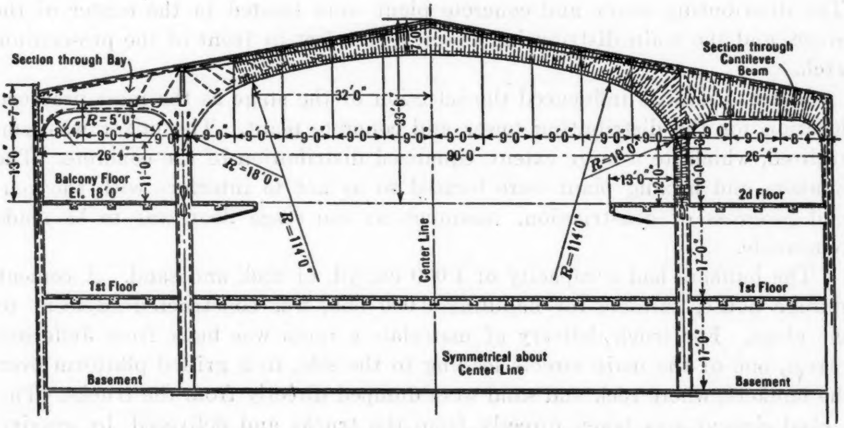

FIG. 16.—CROSS-SECTION OF BANQUET HALL, SHOWING CONTINUOUS ROOF GIRDERS.

PAVILION ROOF GIRDERS

The continuous reinforced concrete girders, supporting the Banquet Hall roof, were designed as a part of a fixed frame (Fig. 16). The soffits of the roof girders (Fig. 4) were curved to represent Saracenic arches, which added to the depth of the girders over the interior columns where large negative moments existed, and presented a very pleasing architectural appearance. It will be noted that the balcony cantilever girders tended to reduce the moments in the columns. The rear, or wall, columns were required to resist an uplift. The anchor arms of the continuous girders were projected through and above the roof (Fig. 16), being hidden from the interior.

FOUNDATIONS

The footings were located in a tight gravel stratum. A maximum of 8 000 lb. per sq. ft. was adopted as the safe bearing pressure. However, this maximum pressure was modified. The foundation having the largest ratio of live, to dead, load was selected and proportioned for the combined dead and live loads. The dead load on this footing was divided by the area thus formed, and the new reduced unit pressure was used for the dead loads on the foundations.

Considerable economy was obtained by combining the exterior retaining walls with the continuous footings, particularly under the east exterior wall of the stage. The area of steel for resisting the tension stresses, due to large

moments in the footings, was reduced to a great extent, as the advantage of a relatively much larger moment arm was obtained by locating the positive moment steel in the upper part of the retaining wall. This is practically monolithic with the footing. The spread, or T-shape, of the footing provided adequate compression area.

Construction Plant

The contractor's construction plant and system of operation were of unusual interest. The concrete had to be distributed over an area of about 3 acres, which necessitated a gravity distributing system and central mixing plant. The distributing tower and concrete plant were located in the center of the stage, and the main distributing tower, a few feet in front of the proscenium arch.

Various reasons influenced the selection of the stage as the most practical location for the distributing tower and concrete plant. The mixer was centralized, which, to a large extent, equalized distribution of the concrete. The bunkers and mixing plant were located so as not to interfere with the general progress of construction, inasmuch as the stage floor was to be made removable.

The bunkers had a capacity of 1 000 cu. yd. of rock and sand. A cement storage house, suitable for handling 3 000 bbl., was constructed adjacent to the plant. For truck delivery of materials a ramp was built from Jefferson Street, one of the main streets leading to the site, to a grilled platform over the bunkers, where rock and sand were dumped directly from the trucks. The sacked cement was taken directly from the trucks and delivered, by gravity chutes, to the storehouse.

Sand and gravel were fed from short spouts along the lower side of the bunkers into regular concrete buggies, which were wheeled an average distance of 20 ft. to the mixer hoppers, and dumped. The concrete aggregate was measured by volume in the concrete buggies. It required three buggy men to feed each mixer when operating at full capacity. The mixing water was controlled by an automatic water tank.

To prevent interruption, due to possible break-downs in the construction plant while casting important structural units for which continuous pouring was necessary, a duplicate plant was specified and installed. This consisted of a two-compartment hoisting tower to which was attached two 1-yd. sliding distributing hoppers and two 1-cu. yd. Smith mixers. The main distributing tower was 238 ft. high. Two tail towers, each 100 ft. high, were built about 300 ft. each from the main hoisting tower; a $1\frac{1}{4}$-in. plow steel cable, 1 500 ft. long, was laid over the hoisting tower to each of the tail towers to support the Lakewood chutes for distributing the concrete. The duplicate plant, in addition to its protection in case of an emergency, greatly facilitated and expedited the general construction, as both mixers and hoists were kept in use most of the time.

Concrete mixed in the proportion of 1 part cement to 6 parts aggregate was used throughout, except in the stage roof arches, columns, and roof arches of the Banquet Hall. All roof arches were cast with a 1:4.5 concrete, and the columns supporting the roof arches of the Banquet Hall were poured with a 1:3 mix.

The strength of the concrete was controlled by mechanical analysis and by slump and compression tests. For the 7-day compression tests, 1: 6 mix, the average of 119 tests was 1 155 lb. per sq. in.; and for 190 tests at 28 days, 1 953 lb. per sq. in. The following is the average of the slump tests: Walls, $5\frac{1}{2}$ in.; footings, columns, beams, and slabs, 5 in.

The capacity and design of the concrete mixing and distributing system were determined largely by the specifications covering the contractor's construction plant, as follows:

"The Contractor shall submit detailed plans of the construction plant, for approval by the Architects (particularly regarding its adequacy and efficiency of operation). The Contractor will be required to install a concrete mixing plant of sufficient capacity, and in duplicate, to insure continuous pouring. The use of a gravity distributing system will be allowed, provided the water content of the concrete, herein specified, is not exceeded, and that the concrete shall not be poured directly into the forms from the chutes, excepting in thick walls, large footings, or roof arches.

"Floor hoppers and concrete buggies shall be used in conjunction with the chuting system."

Forms

The specifications covering the design and erection of form work, contain several paragraphs which represent considerable thought and experience. These specifications read in part:

"In proportioning forms and centering concrete shall be treated as a liquid of its full weight for vertical loads and three-fourths of its weight for horizontal pressures.

* * * *

"Timber supports must be designed so that the number of joints in side-grain compression shall be reduced to a minimum on account of the low bearing value of timber across the grain.

"Care must be taken to prevent lateral displacement of vertical posts due to radial pressures from arch rings, by providing proper longitudinal bracing.

"Vertical supporting members, posts, struts, etc., must have butt-joints or end grain bearing and fish plates. Fish plates shall not be permitted to transmit vertical loads. Scabbing of horizontal supporting members to vertical supports will not be permitted.

* * *

"An additional rise in the centering of arches of $\frac{1}{800}$ of the span shall be provided for shrinkage and settlement of the forms and supports.

"In constructing centers for arches no reliance shall be placed on spiking, but all main members shall be bolted together at joints.

* * * *

"The design and construction of all form work shall be subject to the approval of the Engineer, and no concrete shall be deposited on or in said form work prior to such approval; but such approval shall not be deemed cause for claims for damages or extension of time in case of failure of said form work, nor will it relieve the Contractor from furnishing forms which will produce acceptable work. All such approved plans and descriptions of said form work shall thereupon become a part and parcel of these plans and specifications."

Fireproof Curtain

The asbestos stage curtain is one of the largest in the world, weighing approximately 9 tons. It consists of a rigid steel frame composed of six hori-

zontal latticed trusses 14 in. deep at the center and braced vertically by latticed angle struts extending the entire height of the curtain.

The steel curtain frame is covered on each side with wire-woven, 95% pure, asbestos cloth, weighing 3 lb. per sq. ft.* and having wire inserts. The cloth is fastened to the frame by half round strips, which, in turn, are secured with tap screws, 12 in. on centers.

The curtain is operated on ten 20-in., roller-bearing, machine-turned, cable sheaves and two 24-in. machine-turned head-blocks, set on large roller bearings. A 10 h.p. motor for raising and lowering the curtain is located in the basement. The curtain is equipped with an automatic emergency closure control. In case of fire, it can be dropped by gravity from either side of the proscenium opening by the melting of a fusible link or by the slashing of a small cut line, which automatically releases the curtain. A large oil dash-pot prevents excessive velocity near the floor. The emergency controls are interconnected with electrical door operators for opening the large ventilators over the stage.

The frame was designed to prevent side-swaying, sagging, and bellying, due to unequal air pressure on the two sides, and to withstand a horizontal lateral pressure of 5 lb. per ft., with a factor of safety of 2, without binding on the smoke-pocket guides. The lateral pressure is based on a temperature of 750° Fahr. on the Auditorium side and 1 600° Fahr. on the stage side, with 15-min. exposure.

The curtain has an unbalanced weight sufficient to cause it to drop by gravity with a frictional factor of 1 in 32 sec. As designed, the whole curtain descends as a unit, making its entire unbalanced weight effective in dropping.

The stage scenery equipment (Peter Clark System) consisted of 100 counterweighted sets, operated on T-bar tracks, about 4 in. on centers. A bridge located along, and cantilevered from, the rear wall of the stage provides the facilities necessary for painting the exceptionally large scenes.

The Acoustics

The acoustics of the Auditorium was the subject of much thought. Detailed study was given the correlation of size and shape to prevent objectionable reverberation. The selection of lime plaster for finishing the interior wall surfaces assisted, to some extent, in absorbing the sound. The great hung canopy and the surrounding ceiling, with its numerous broken surfaces, were treated so as to subdue undesirable reflection without losing any of its architectural beauty. All the seats were upholstered and the aisles carpeted. Not only was this in keeping with the luxuriousness of the appointments, but it also served further to prevent acoustical defects.

As a result, normally, stage programs can be heard without difficulty in the rear seats of the balcony, a distance of 235 ft. However, this was considered too great a distance for the average human voice, or subdued music, to carry successfully. Consequently, it was deemed advisable to provide an amplifying system.

* Same as tested by the U. S. Bureau of Standards, Washington, D. C., in its recent curtain tests.

Public Address System

In keeping with the high standards utilized throughout this edifice, it was desired to obtain the most comprehensive and complete system available, as well as one that would faithfully amplify, transmit, and reproduce the program. To meet these requirements, a Public Address System was installed by the Graybar Electric Company.

It is used for many and varied functions. In addition to reproducing speech and music from the stage of the Auditorium, it provides a means for "piping" the program from one room to another, thus caring for overflow meetings. This Public Address System is also used for amplifying in the large ballroom, for reproducing radio programs when used in conjunction with a radio receiving set, for paging throughout the building, and for cab calls.

To provide these features, it was necessary to install in and about the Temple a total of 46 projectors or horns, 25 microphone outlets, and 24 observer stations. A special control room, adjacent to the Exhibition Hall, was used for housing the amplifying and radio receiving equipments and auxiliary apparatus. The microphone outlets were installed at strategic points throughout the building for conveying, to the amplifying system, such programs as it is desired to reproduce throughout the building itself, or to transmit over the air through a radio broadcasting station.

Similarly, observer positions were provided for watching the performance of the loud speaker equipment in each particular room. A telephone circuit, between these positions and the control room, enables the observer to advise the operator if corrections are required. The power used for operating this system is obtained from the regular alternating-current house supply and is adjusted to the proper voltage by a vacuum tube rectifier system.

Since the Auditorium is thus equipped, it is not necessary to request front seats in order to enjoy the program. There are no panels to hinder the view, the projectors being carefully concealed in the proscenium arch 42 ft. above the stage. They reproduce the program so that those in the rear seats can hear as well as those in front.

Heating and Ventilating

The heating system consists of three Birchfield welded, low-pressure boilers, set in battery with Fess oil burners. Each boiler has a capacity of 30 000 sq. ft. of radiation. The Auditorium and Banquet Hall are heated and ventilated by a forced-air circulating system.

The majority of people who visit this edifice are so impressed with the lavishness of its appointments, that they are apt to take for granted the perfect air conditions, that prevail, without realizing the great engineering skill that made such conditions possible. The average layman would be astounded to know that, in order to insure perfect ventilation, it is necessary to handle 43 254 000 cu. ft. of air per hour. This enormous quantity is distributed by 12 Sturtevant ventilating fans, of various sizes, located at strategical points throughout the building. To the Auditorium alone 14 838 000 cu. ft. is supplied, and correspondingly large quantities are furnished to dressing-rooms under the stage, the Banquet Hall, basement, etc.

The Auditorium is ventilated by the up-draft system (Fig. 17), wherein the air is forced into plenum chambers under the seats and thence through mushroom outlets into the room. An exhaust air system, with outlets in the ceiling under the balcony and in the ceiling of the Auditorium, draws the vitiated air up past the breathing zone and exhausts it to the outside. The up-draft system for a building of this type is much superior to the so-called down-draft system, wherein the air enters the ceiling and is sucked out under the seats. In the up-draft system the laws of nature are utilized as the heated air rises, which assists the exhaust system in its work. A very important point in favor of the up-draft system is the quick elimination of smoke, especially during the Shrine ceremonials.

The incoming fresh air is heated by large units of Aerofin heaters which are automatically controlled by thermostats set in the various rooms. The stage is amply heated to compensate for differences in temperature between the Auditorium and the stage area, thus eliminating the great drafts that usually occur when the curtain is raised.

Decoration

The architectural treatment of the entire structure is Saracenic, with the Moorish motif predominating. Rare beauty marks the ceiling of the Auditorium (Fig. 2) with its jeweled tapestry canopy reflecting the blue of an Arabian sky. The tapestry effect on the huge plastered canopy with its large ropes (9 in. in diameter) of cast plaster, is so realistic that many have believed that this immense ornament was constructed of real tapestry and fibered ropes.

The whole effect of the interior of the Auditorium is heightened and magnified by a gloriously beautiful envelope of Oriental filigree work and daring color schemes (Fig. 5), which is truly Saracenic in design. The Saracens were celebrated for their decorative art, in which the Oriental love for display and color demanded a certain luxuriousness, that is reflected through the interior treatment.

A great deal of decoration was applied directly to the concrete beam and wall surfaces, which were tinted and stenciled, producing a very striking and interesting effect, particularly on the roof arches in the Pavilion (Fig. 4).

The concrete exterior of the entire building was treated with California stucco of a light buff color, applied with an air brush and producing a durable coating of a pleasing uniform texture. The exterior ornamentation consists of polychromed tooled synthetic limestone, composed of crushed limestone and Portland cement.

Electric Lighting

In the center of the suspended canopy is hung a magnificent chandelier, 22 ft. high and weighing approximately 5 tons (Fig. 2). It contains 500 electric bulbs, arranged to produce 64 color combinations, all controlled by dimmers on the stage switchboard. There are fifty-one 1 320-watt circuits, nine 150-watt lamps to each circuit, and three emergency circuits. To each of the four colors—red, white, blue, and amber—there are twelve circuits.

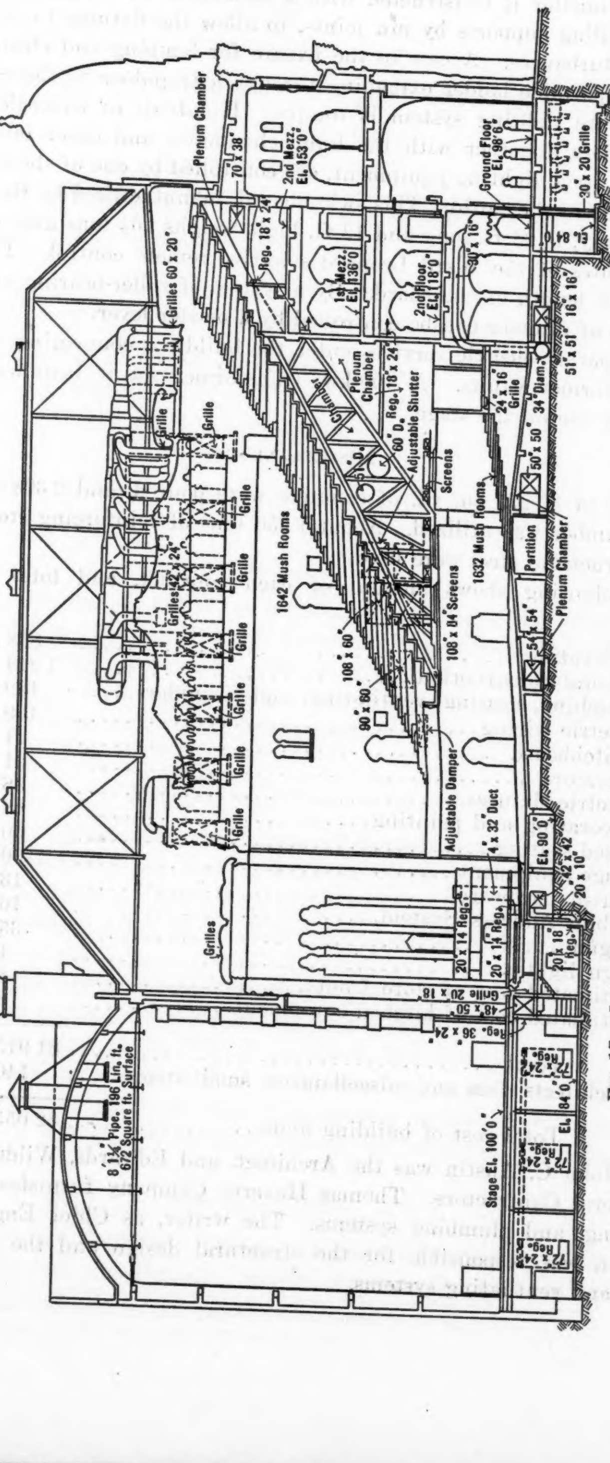

FIG. 17.—LONGITUDINAL SECTION OF AUDITORIUM, SHOWING UP-DRAFT SYSTEM OF VENTILATION.

The chandelier is constructed with a structural steel frame connected to the steel ceiling supports by pin joints, to allow the fixtures to swing during seismic disturbances. Access to the fixture for lamping and cleaning is provided by a portable ladder extending through a trap-door in the ceiling.

The entire lighting system is unique. Hundreds of concealed lights of varying colors, together with the large chandelier and other minor fixtures and all the stage lighting equipment, are controlled by one of the largest stage switchboards in the world. This switchboard, manufactured by the Hub Electric Company, is 28 ft. long and 12 ft. high, weighs 16½ tons and contains 298 dimmer plates of the Ward Leonard selective remote control. The dimmers run the full length of the board. By a system of roller-bearing mountings, a full length of dimmer can be controlled by a master lever.

Two separate electric services enter the building, connecting to two separate transformer vaults. A 100-kw. transformer set is installed for direct current for use on the stage.

Miscellaneous

A total of 30 275 cu. yd. of concrete were poured, and 2 308 000 board-ft. of form lumber was utilized. About 1 550 tons of reinforcing steel and 1 100 tons of structural steel were used.

The following shows amount of each contract, and total cost of the building:

Excavating	$38 520.00
General construction	1 214 452.00
Plumbing, heating, ventilating, and sprinklers	190 000.00
Electric wiring	109 532.00
Switchboard	41 939.00
Elevators	21 570.00
Electric fixtures	38 641.00
Decorating and painting	68 000.00
Fixed seating	50 000.00
Stage equipment	60 984.00
Finish hardware	13 140.00
Public Address System	16 246.00
Organ	35 000.00
Sign lights	1 295.00
Estimated cost of store fronts	8 000.00
Estimated cost of kitchen equipment	8 500.00
Total	$1 915 819.00
Architects' fees and miscellaneous small items	140 000.00
Total cost of building alone	$2 055 819.00

Mr. John C. Austin was the Architect, and Edwards, Wildey and Dixon, the General Contractors. Thomas Haverty Company furnished the heating, ventilating, and plumbing systems. The writer, as Chief Engineer for the Architect, was responsible for the structural design and the design of the heating and ventilating systems.

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METHODS USED IN THE CONSTRUCTION OF TWELVE PRE-CAST CONCRETE SEGMENTS FOR THE ALAMEDA COUNTY, CALIFORNIA, ESTUARY SUBWAY

By Alvin A. Horwege,* Assoc. M. Am. Soc. C. E.

Synopsis

This paper describes in detail the construction of the twelve pre-cast concrete segments of the Estuary Subway between Oakland and Alameda, Calif. It includes a brief statement of the nature of the work and of general conditions, a description of the mixing plant, distributing system, and forms, and the method used in pouring the different sections of a segment.

Purpose of Construction

The site of the Oakland-Alameda Estuary Subway was formerly occupied by the railroad bridge of the Southern Pacific Railroad Company and used as a suburban connection between Oakland and the Alameda Pier, and is within several hundred yards of the highway and street-car bridge connecting Oakland and Alameda.

Because of the heavy increase of the shipping in the Oakland Estuary these bridges had become a serious menace to the traffic in the Upper Estuary and, at the solicitation of the War Department Engineers, Alameda County began an investigation which resulted in the approval of plans and the voting of bonds for the construction of an estuary subway which will eliminate both bridges and give the Upper Estuary Harbor an unobstructed channel for shipping.

Nature of Work

Early in 1925, the California Bridge and Tunnel Company was awarded a contract to construct the Estuary Subway between Oakland and Alameda,

Note.—Written discussion on this paper will be closed in April, 1928.

* Supt., California Constr. Co., San Francisco, Calif.

Calif. In general, the work consisted of two open-cut approaches totaling 930 ft.; two portal buildings totaling 136 ft.; a rectangular section on the Oakland side, 342 ft. long; a transition section, 37 ft. long, connecting the rectangular to the circular section; a circular section built in place on the Oakland side, 578 ft. long; and twelve pre-cast concrete tubes, each 203 ft. long, to be built in Hunters Point Dry Dock, floated across the Bay, sunk in place, and joined under water.

General Conditions

Time was an important element in the construction of these tubes. The Bethlehem Shipbuilding Corporation was reluctant to permit the dock to be tied up a reasonable length of time, from the standpoint of economy to the contractors. The rental of the dock was limited to six months, after which a severe penalty was to be added to the contract rental. The result was that the contractors were forced to provide a rather elaborate plant, considering the quantity of concrete to be poured—29 000 cu. yd.—and to use steel forms throughout. To permit rapid erection, three full shifts of men were employed during the construction period.

Delivery of Material

Practically all the material, except the cement, was delivered in barges. (See Fig. 1.) The cement was hauled from the railroad, a distance of 2 miles, in trucks as it was used. At first an attempt was made to store a reserve supply on the job, but because of rehandling, this proved more costly than to haul as it was needed.

The rock and sand were delivered in barges of 400-ton capacity and unloaded with an electric hoist and steel derrick equipped with a 60-ft. boom and a 1-yd. clam-shell bucket. The material was dumped into a receiving hopper at the end of the dock, from which it was conveyed by a system of 24-in. belt conveyors to a storage pile. These piles were confined by wooden retaining walls filled with the material to give them stability. The first conveyor was 94 ft. long, and dumped on the second conveyor, 114 ft. long, at an angle of about 15°; and this, in turn, delivered to a movable distributing conveyor, 40 ft. long, which spread the material in piles, about 75 ft. long, at right angles to the belt conveyor. The sand was segregated from the rock and, of course, could not be conveyed from the barge at the same time as the rock.

Mixing Plant

Adjacent and parallel to the stock piles a ramp, 20 ft. high, was erected, from which a crane, fitted with a clam-shell bucket, fed the hoppers on top of a two-story mixing house. (See Fig. 1.) Two 1-yd. concrete mixers were placed on the ground floor of the mixing house. Batchers for the rock, and inundators for the sand, a separate set for each mixer, were placed on the second floor. The inundators, because of the almost uniform moisture content of the sand received, and to permit more speed of operation, were used as batchers.

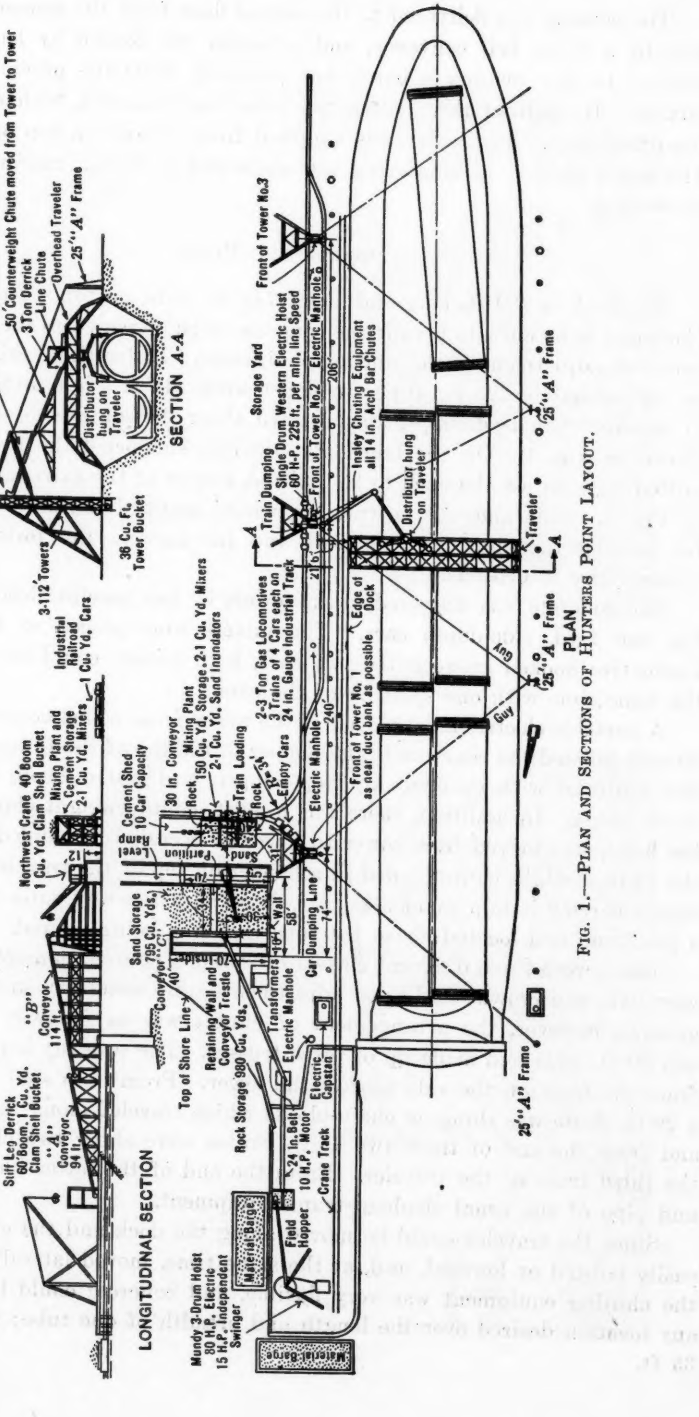

FIG. 1.—PLAN AND SECTIONS OF HUNTERS POINT LAYOUT.

The cement was delivered to the second floor from the cement house adjacent by a 24-in. belt conveyor, and a hopper was loaded by hand above the mixer. In this manner a batch was prepared while the previous batch was mixing. By pulling three ropes the mixer was charged, both mixers acting simultaneously. The water was supplied from a tank on top of each mixer. The usual method of measuring was employed so that a uniform mix could be assured.

Distributing Plant

The dock is 750 ft. long and about 110 ft. wide, on top. The distributing plant had to be built to permit the concrete to be delivered to the dock without too much adjustment of the chuting equipment. To facilitate this, at intervals of approximately 225 ft., three wooden towers, 112 ft. high, and large enough to accommodate 1-yd. skips, were erected along the south edge of the dock as shown in Fig. 1. On the land side stiff-legs supported the towers and permitted guys across the dock to be omitted, except at the extreme top as shown in Fig. 2. This allowed the traveler, which spanned the dock and was used for holding the chuting equipment and for moving the forms, to operate without any interference.

The concrete was delivered to the towers by two gasoline locomotives hauling two 1-yd. side-dump cars. The mixers were placed so that with the locomotive hooked between the two cars, both mixers could be discharged at the same time with one spotting of the cars.

A portable electric hoist placed at the tower from which concrete was being poured, hoisted the concrete to the hopper at the top of the tower. Each tower was equipped with its own skip, top hopper, and a steel boom with 40 ft. of 12-in. chute. In addition, there was a 40-ft. counterweight chute which, like the hoist, was moved from tower to tower as occasion demanded. The end of the 40-ft. straight chute landed in a hopper on top of the traveler, from which chutes carried it to a wooden hopper, fitted with two-way gates operated from a platform, and located above the tube that was being poured.

There were three different classes of sections poured, namely, invert, roadway slab, and crown; and some adjustments were necessary in each case. In general, however, the arrangement of chutes was as follows: The traveler was 20 ft. wide and made up of three trusses. The two-way hopper was slung from the truss on the side nearest the tower. From each side of the hopper, a 20-ft. chute was slung on chain-blocks which traveled along the center truss, and from the end of these two 15-ft. chutes were slung two chain-blocks on the third truss of the traveler. From the end of the latter hung the hopper and pipe of the usual elephant-trunk equipment.

Since the traveler could be moved along the dock and the chutes could be easily hoisted or lowered, and, at the same time, moved laterally to the dock, the chuting equipment was very flexible, and concrete could be delivered to any location desired over the length and breadth of one tube; that is, 203 by 35 ft.

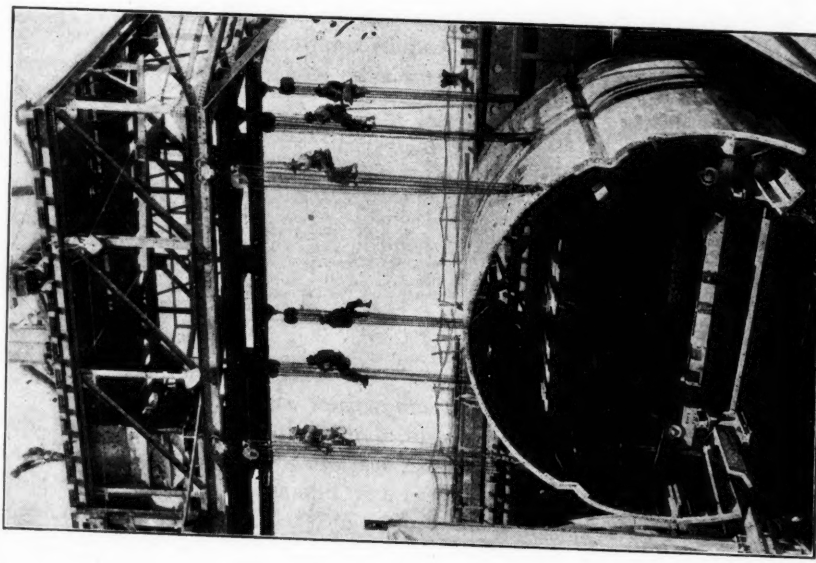

FIG. 3.—VIEW SHOWING SECTION OF INSIDE CROWN FORM BEING MOVED.

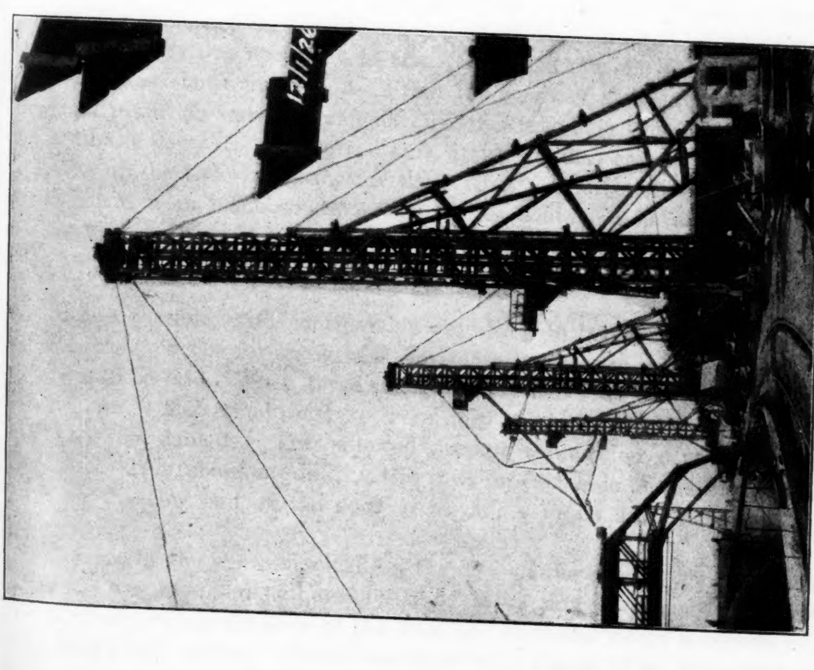

FIG. 2.—CONCRETE TOWERS AND CHUTE SYSTEM.

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Concrete

Table 1 gives the various gradings for the Class "A" mix used in the construction of the tubes, proportioned for 1 cu. yd. of concrete in place and certain characteristics based on a series of tests.

TABLE 1.—Concrete Data.

Grading, n	0.53	0.54	0.55	0.56	0.57
Fineness modulus of sand	5.87	5.91	5.95	5.99	6.03
Cement, sacks	8	8	8	8	8
Sand, in cubic feet	11.19	11.7	11.4	11.2	11.9
Rock, in cubic feet	21.0	21.4	21.8	22.2	22.6
Ratio of cement to voids	1.28	1.26	1.24	1.21	1.19
Ratio of sand to voids	1.14	1.09	1.05	1.01	0.98
Ratio of mortar to voids	1.96	1.91	1.86	1.81	1.76
Density	0.82	0.825	0.833	0.842	0.85

A shrinkage of 3% in setting was allowed, and the proportions produced not less than 1 cu. yd. of concrete in place.

The grading, n (0.55), and the fineness modulus (5.95), were most satisfactory for the conditions and with a slump of from 6½ to 7½ in., the concrete developed a strength of 3 000 to 3 500 lb. in 28 days.

Forms

On the Hunters Point work the inside crown forms were of steel and the outside crown forms had steel ribs and wooden lagging. Wherever possible, forms were mounted on trucks and ran on tracks from one tube to the next, except where it was necessary to transfer them from one side of the dock to the other, as shown in Fig. 3. From the roadway slab and the crown, the original intention was to pour one-third of a tube at a time and the complete tube in the case of the invert. This, it was found, caused too much delay because of the time necessary for the setting of the concrete, and additional steel forms were made where wooden forms could not be used, and wooden forms were built on the job in the other instances.

Preliminary Work

It was possible, with considerable crowding, to lay five tubes in the dock at one time, except for those with horizontal curves, in which instance only four could be laid. When three straight tubes had been completed, they were floated out. The remaining two were only partly finished and were flooded without any damage. This allowed prompt resumption of the work after flotation. The tubes were laid in two rows on each side of the dock, with one tube at the bow end of the dock at an angle to its axis, the others being parallel.

Because of the enormous load on the floor of the dock, 5 000 tons for each tube, the weight was distributed by 12 by 12-in. timbers placed longitudinally on 2-ft. centers, on the floor of the dock and shimmed where a depression did not permit full bearing. Across this layer were placed 12 by 12-in. timbers 38 ft. long, spaced to conform with the invert forms and, roughly, about 3½-ft.

centers. These timbers were leveled carefully at a fixed elevation so that calculations for the setting of forms were the same for all tubes. Great care was taken to provide uniform bearing in this case and thus prevent excessive settlement. Fig. 4 shows the subsequent stages of construction work herein described.

Saddles

The center portion of the tube for a width of 10 ft., 1½ in. on each side of the center line, was supported on wooden saddles which remained in place until the tube was floated. The center section was made from 4 by 16-in. timbers, 7 ft. long. Bolted to these at one end and to 6 by 12-in. posts at the other, were secondary saddles made from 4 by 12-in. timbers. They were also supported twice in the middle of the span with 4 by 6-in. posts. All saddles were cut to conform with the outside curvature of the completed tube. The secondary saddles were unbolted just prior to floating the tubes and were pulled from under the tube afterward. This was necessary in order to provide clearance for the end bracket at the end of the tubes. Fig. 5 is a view of a lower cradle and outside form, showing the method of erection.

Outside Invert Forms

On each side of the wooden saddles were placed the outside invert forms. These were steel ribs built as cantilevers and tied together into 19½-ft. sections. There were seven ribs with wooden nailing strips fastened to curved angles at 3 ft. 3-in. centers. Between these were wooden ribs, 2 in. wide, to reduce the span. (See Fig. 5.) Each invert section was separated from the next by 22-in. spacing which permitted them to be swung, in the case of the horizontal curves, without interference.

After these invert forms were set in place with cranes, they were lined and leveled by hard-wood wedges on top of 8 by 12-in. longitudinal timbers and braced with struts to the floor of the dock. The lagging of 2 by 12-in. material was next laid on top of the invert forms and the wooden saddles up to the horizontal diameter. As this lagging served a double purpose, that of the outside forms and the protection for the water-proofing, and remained on the tube, it was not nailed to the form ribs except to draw a warped end down. Outside invert forms were provided for two complete tubes.

End Brackets

At each end of the tube the design specified an end bracket, which consisted of a square block of concrete through which the circular portion of the tube passed, as shown in Fig. 6. These served a double purpose; that is, as bases for landing the tubes in place on concrete blocks, and for joining the tubes. The forms for the end brackets were of steel, except for the wood bottom which remained in place until the tube floated off it. There were four complete forms for the invert; two inside rings; two outside sets of plates up to a little above the horizontal diameter; and one form for the top outside and the end face.

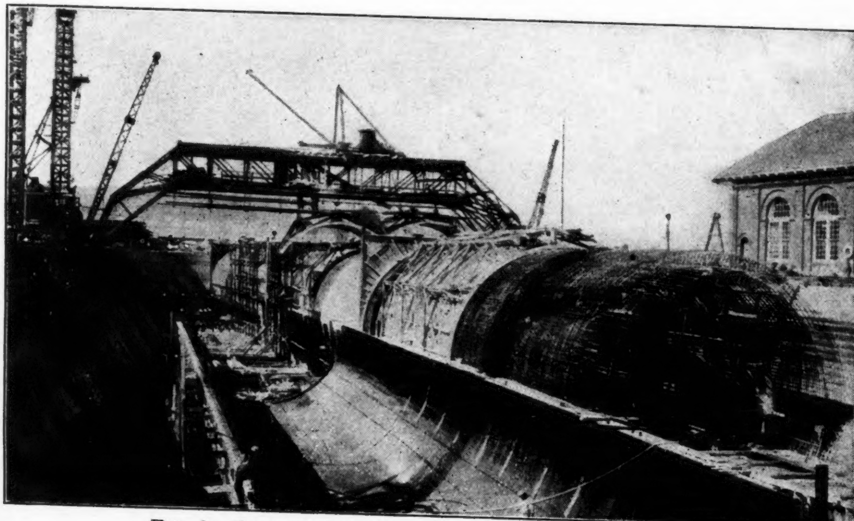

FIG. 4.—VIEW SHOWING VARIOUS STAGES OF CONSTRUCTION.

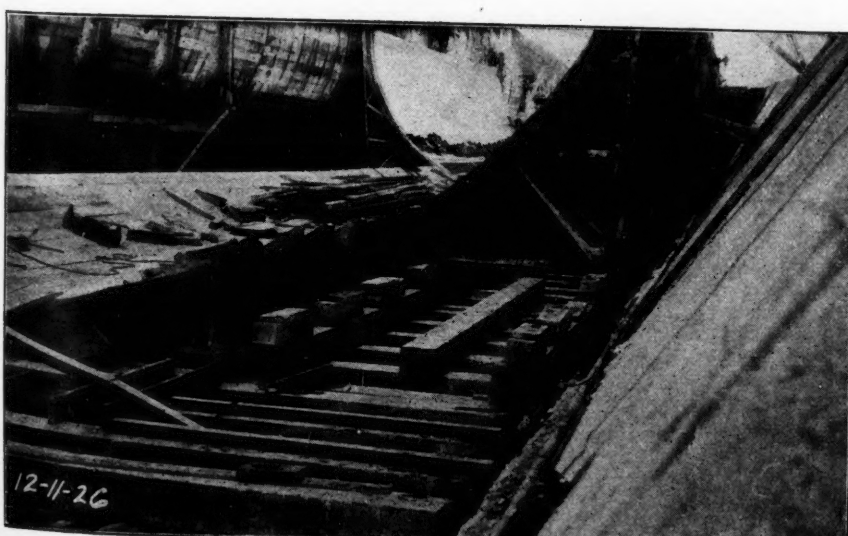

FIG. 5.—VIEW OF LOWER CRADLE AND OUTSIDE FORM.

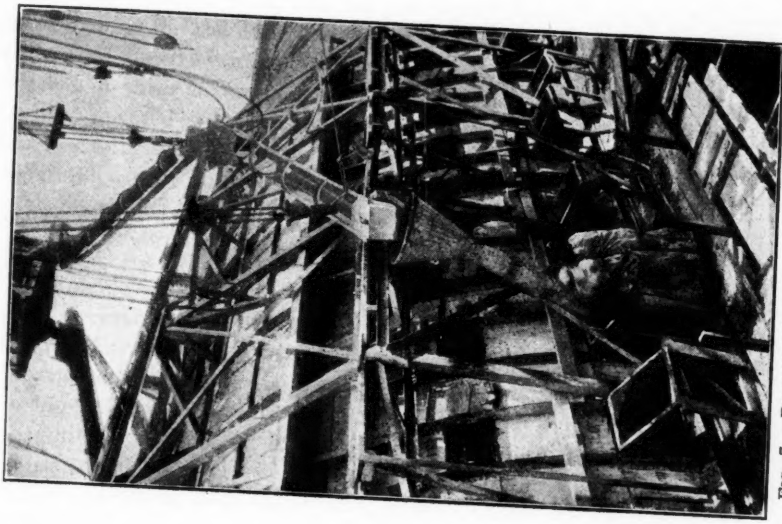

FIG. 7.—POURING CONCRETE IN CROWN SECTION.

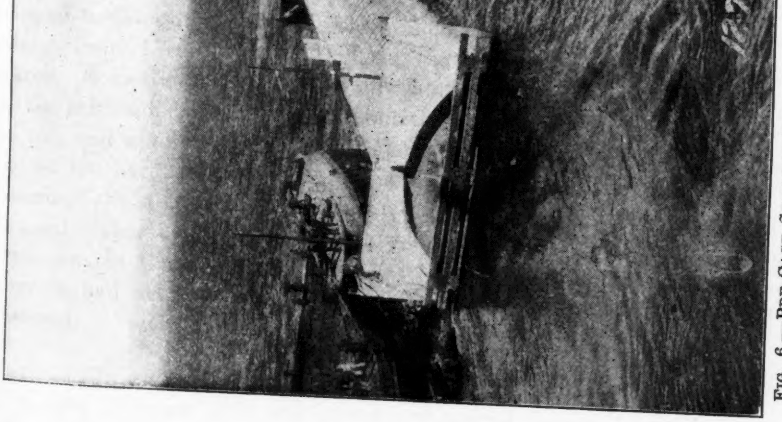

FIG. 6.—PRE-CAST SEGMENT LEAVING HUNTERS POINT DOCK.

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Invert Water-Proofing and Steel

After the invert lagging and the end bracket invert forms were placed, the water-proofing for the tube, below the horizontal diameter, was laid on the lagging and ended by metal flashing in the end brackets. The water-proofing consisted of three layers of cotton fabric in hot asphalt. The bottom 20 ft. was next covered with a 1-in. layer of grout to protect the water-proofing from the reinforcing steel that was next placed. The steel was set on 2-in. concrete blocks 2½-in. high. It was found that the weight of the steel forced the blocks through the water-proofing unless it was protected.

Inside Invert Forms

The inside invert forms were made of steel and were hung to the outside forms by means of steel brackets and bolts. The bolts passed through the water-proofing, the holes being later patched. They extended from a point approximately level with the bottom of the roadway slab to a point 8 ft. from the center of the tube. This left an opening 16 ft. wide in the bottom of the tube, which was supposed to be screeded to conform to the inside curvature. Because it was necessary to pour the concrete rather wet so that it would flow properly around the network of steel, it was found that the steel forms had to be supplemented with wooden sections 4 ft. wide to reduce the opening to 8 ft. Both the steel and wooden forms were securely fastened with ⅜-in. bolts to the reinforcing steel and rested on the 2½-in. concrete blocks on top of the steel. Four of the tubes had an 8-in. longitudinal wall under the center of the roadway slab. A projection, 16 in. high, was formed in this opening. Otherwise, the concrete was screeded and finished smooth while being poured. The form sections were 22 ft. long and enough for one complete tube were provided.

Invert Concrete

The entire invert, which contained about 715 cu. yd., was poured continuously and required about 11 hours to complete. For the first half of the tube, the bottom was filled to the lower part of the form. The chutes were then changed to the outside and the sides filled to the top. At the same time the concrete would run forward and fill the bottom ahead of that previously poured. Sometimes it was found advisable to switch back and fill the bottom for the remainder of the distance, but the bulk of the concrete was poured at the top and allowed to run down along the outside forms. Before the top was set too hard it was worked by hand to conform to the longitudinal joint prescribed by the County Engineer. This included a key to which was fastened either a copper or galvanized water-stop 10 in. wide, one-half of which was in the invert concrete. The forms were stripped as soon as the concrete had set sufficiently and all irregularities were smoothed off and plastered.

Roadway Slab Forms

Within 10 hours after the invert was poured four rails were laid on the concrete on longitudinal stringers properly spaced and braced to take the trucks

of the roadway forms. These forms were then rolled out of the previous tube on to this track and jacked up to grade. The forms consisted of three sections, 21 ft. 8 in. long, mounted on eight trucks and made of steel. Between them were suspended two sections $16\frac{1}{2}$ ft., made of wood, on the job, and sheathed with No. 10 gauge plates. Each section was separated along the center line and, in the case where the center wall was under the road slab, vertical forms with suitable turnbuckles for adjustments were suspended to these forms. In other cases, the opening between them was covered with a No. 10 gauge plate. On the outside edge were hinged wing forms which were swung up against the invert concrete by turnbuckles and then securely held by wooden struts. The wooden forms were held by wooden posts and wedges.

As soon as the forms were set to grade the road-slab steel was placed and side longitudinal forms of wood set in between the steel were built. Screeds, all the interior fixtures, and bulkheads at the end, were placed and the concrete for one-half the tube was poured. This amounted to about 158 cu. yd.

The forms were allowed to stand for 48 hours when they were collapsed and moved either to the other half of the same tube, or to the next tube.

Reinforcing Traveler for Crown Steel

Twenty-four hours after the road slab was poured, two 50-lb. rails were laid on top of longitudinal stringers. The reinforcing traveler was then moved on to the road slab and jacked into place. This traveler consisted of four sections, each 20 ft. long. Three sections were mounted on trucks, and the fourth was suspended by 12 by 12-in. stringers between two of those on the trucks. This arrangement was used only when moving; when spotted, the fourth section was jacked up directly from the road slab. The others had jacks mounted on the trucks. All were provided with turnbuckles for spreading to the proper width, and collapsing after the steel was in place.

Each section was made up of a steel bed-frame, on which were mounted three wooden lateral trusses. Running longitudinally were metal angle-bars at intervals of about 10 ft., which later were supplemented by wooden members between, and all built to conform to the prescribed radius of the reinforcing steel, that is, 16 ft. $2\frac{1}{2}$ in. Each section was separated about 5 or 6 ft. according to the needs and closed with spliced angles. Fig. 4 is a view of the reinforcing steel traveler with crown steel in place.

When the traveler was set to grade and spread to the proper width, the arch or crown reinforcing steel was placed on it. As in the invert, there were two rings of this tied together with longitudinal steel and also to the invert rings which protruded from the invert concrete. The ring steel varied from $\frac{3}{4}$ in. to 1 in., according to the tube, and was spaced on 12-in. centers. The longitudinal steel was $\frac{3}{4}$ in. throughout and was spaced on 6-in. centers.

Outside Crown Forms

The original outside crown forms were made up of three sections each 21 ft. long. Each consisted of two pairs of trusses hinged at the top and heavily cross-braced between. To this frame was bolted 3 by 12-in. wooden lagging.

When the length was increased to make possible the pouring of one-half the tube instead of one-third, two more 15-ft. sections were built of similar design. Four rows of doors were provided on each side through which the concrete could be poured and a 10-ft. space, where there was no lagging, was left at the top.

The bottom rested on top of the outside invert forms at the horizontal diameter and was bolted securely to it. Just above this point holes were provided to allow a row of 1¼-in. bolts to pass to the inside crown forms. These were spaced about 4-ft. centers and were the only lateral support for these forms. When these forms were set, hook-bolts were passed through holes provided at intervals and hooked to the reinforcing steel and tightened. The reinforcing traveler was then collapsed and moved out to the next set up. Fig. 4 shows a view of an outside crown form in place; also, a section of the outside crown form being lifted and moved with the gantry.

Inside Crown Forms

The inside crown forms were next moved into place under the reinforcing steel which was suspended from the outside crown forms by the hook-bolts. These forms consisted of three sections, 23 ft. long, each mounted on trucks, as shown in Fig. 3. Two sections, 15 ft. long, were built in San Francisco, Calif., when the "pour" was increased, and were suspended between the original sections. The first three sections were provided with jacks for raising and steamboat ratchets for spreading to the proper width. The others had no such mechanism, but when bolted to the first were rigid enough to follow the others. All were hinged at the top.

The general design consisted of a heavy bed-frame mounted on trucks with two heavy pairs of trusses rigidly cross-braced and covered with No. 10 gauge plates. A heavy waler ran longitudinally just above the horizontal diameter to take the 1¼-in. through bolts from the outside crown forms. Near the bottom was a hinged wing form. When moving, this was hooked up and kept there until the form was set at the proper grade, and spread when it was lowered to a sloping wooden filler strip. Struts were then placed from the track stringers to the base of these wing forms and wedged, and the jacks were slacked off. The forms then rested on the sloping filler pieces and became a rigid self-contained unit.

After the 1¼-in. through bolts were set, the interior fixtures placed, and a bulkhead was provided at the center of the tube, the forms were ready for concrete. Horizontal and vertical curves were formed by means of tapered filler pieces between the sections of forms. These were carefully calculated and the forms had to be spotted accurately to make them conform to the required curvature.

End Bracket Form

As soon as possible after the outside crown forms were placed, the back top of the end bracket form was set. For ease in handling, this was divided

into three pieces and the reinforcing steel for the portion was placed against it. The front section was next placed and the form was ready for pouring.

Crown Section

The crown section amounted to about 700 cu. yd., for one-half the length of the tube and was poured in about 12 hours. This was divided into three operations. The first was through the lower row of doors. (See Fig. 7.) The steel was spread to allow the hopper chutes to be inserted into the center of the wall. These hopper chutes had a bend in the spout so as to turn the concrete in a vertical direction and prevent the rock from hitting the steel and separating from the mortar. There were 13 to 14 chutes on each side, placed directly opposite in pairs. Ten men on a side were stationed between the forms to tamp and spread the concrete while it was being poured. When the forms were filled to the row of doors, the hoppers and chutes were raised to the next row, the lower doors were closed, and the process was continued. When the forms were filled to the second row they were closed and the chutes were raised to the top opening and the remainder of the concrete was poured from that point. The top of the end bracket was poured last. Six men with air hammers vibrated the inside crown forms during the pouring.

Forty-eight hours after completing the "pour" the outside crown forms were moved to the next position. The outside invert forms were moved at this time, also. Twenty-four hours later the inside crown forms could be moved.

Water-Proofing and Lagging

As soon as the outside crown forms were moved men with chipping hammers smoothed up the concrete so that the water-proofing would not be injured. Scaffolding was then set up and the upper half of the tube was water-proofed. The lagging, for protection, was then placed and hoop rods every 3 ft. held this in place. Temporary wooden forms were set up against the lagging below the horizontal diameter to hold that lagging against the concrete after the invert forms were removed and before the hoop rods could be placed all around the tube. As soon as the rods were tightened these wooden forms were removed.

Concrete Bulkheads

Bulkheads were built to close the ends of the tubes for purposes of flotation and submersion. Below the roadway slab a heavily reinforced concrete bulkhead was built. Wooden forms were used in this work.

Wood and Steel Bulkheads

Above the roadway slab the construction was varied to suit the pressure. Under high heads 24-in. steel I-beams were set vertically at 1 ft. centers, against a shoulder provided in the end bracket. They were covered with 6 by 8-in. timbers, laid flat, and then sheathed with 1 by 6-in. tongue-and-groove stock. The whole was water-proofed, the edges being run into a keyway pro-

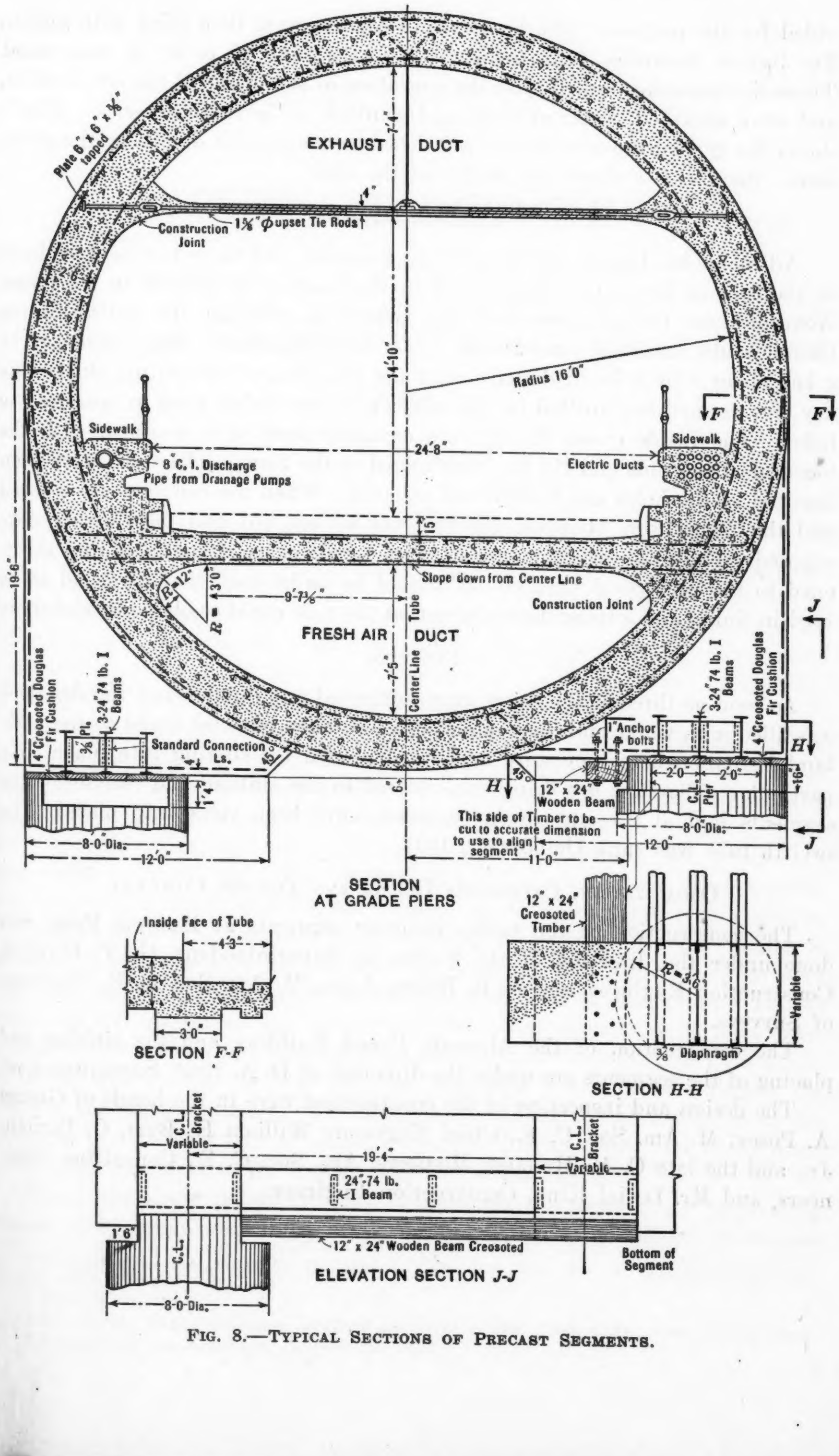

FIG. 8.—TYPICAL SECTIONS OF PRECAST SEGMENTS.

vided for the purpose. The keyway and corners were then filled with gunite. For lighter pressures, 12-in. timbers of depths from 20 to 30 in. were used. These timbers were framed to fit the curvature of the inside of the end bracket, and were sheathed, water-proofed, and gunited, as in the other case. Fig. 8 shows the typical cross-section of a finished segment, with details at the grade piers. Section *F-F* shows the design of the ends.

Interior Detail

All manholes, inserts, niches, electric conduits, and tie-rod stubs were built in the tube as it was constructed and in the location designated in the plans. None of these features presented any difficulties, although the setting of the tie-rod stubs involved considerable labor and attention. They were set in a key about 8 by 8 in. in dimension at the elevation of the ceiling slab. The key was framed and drilled in the mill, great care being used in spacing the holes. The inside crown forms were carefully spotted to conform with the location of the rods and the key was lagged to the form. The stubs were then inserted in the holes and leveled and squared. When the concrete was poured and the forms were stripped, the key was broken out and the threads were exposed. The rods were then set up with turnbuckles at each end and tightened to the prescribed tension. It should be understood that care had to be used in lining and setting the stubs so that the rods could easily be threaded on.

Floating

As soon as three of the tubes were completed and bulkheaded, the dry dock was filled with water and the completed tubes taken out and towed to the Oakland Estuary where they were held until needed for sinking into place. No particular difficulties are being encountered in the sinking and placing of the segments and at present eleven segments have been sunk into place. The twelfth tube was sunk October 29, 1927.

Organization, California Bridge and Tunnel Company

The construction of the twelve concrete segments at Hunters Point was done under the direction of the writer, as Superintendent, G. T. O'Brien, Construction Engineer, and A. R. Baker, Assoc. M. Am. Soc. C. E., Engineer of Surveys.

The construction of the Alameda Portal Building and the sinking and placing of the segments are under the direction of D. A. Root, Superintendent.

The design and inspection of the construction were in the hands of George A. Posey, M. Am. Soc. C. E., Chief Engineer; William H. Burr, C. Derleth, Jr., and the late C. M. Holland, Members, Am. Soc. C. E., Consulting Engineers, and Mr. Locial King, Construction Engineer.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

AUTOMOBILE HAZARD IN CITIES AND ITS REDUCTION

Discussion*

By William J. Cox, Jun. Am. Soc. C. E.†

William J. Cox,‡ Jun. Am. Soc. C. E. (by letter).§—The writer's purpose in the presentation of this paper was not to create a finished document, nor to state a final word. Rather it was to open a new line of thought on the problem of automobile accidents, and to encourage thinking along more fundamental lines as to the basic factors in their occurrence. He feels gratified that the paper has aroused so much discussion.

The writer did not intend to suggest that the automobile accident problem cannot be influenced by other factors than the density of population. He did wish to show that density of population is the basic factor in determining the personal injury hazard of operating an automobile in a city. Other factors, some susceptible to statistical treatment, perhaps, and some not, may exert a modifying influence and should, of course, be given careful consideration in traffic control and accident prevention work; but, basically, the hazard level is a function of the density of population. The writer is satisfied in his own mind that this is the case, and has been pleased to note that most of the commentators on the paper agreed on this point. It was suggested by several of the discussors that more stress should have been placed on conclusions to be drawn from the principle that the paper set forth. It is the writer's feeling that to have done that would have enlarged the scope of the paper unwisely. The aim was, as stated, to call attention to the principle in the hope that it might encourage further work and thought on the part of others as to its applications and implications.

Among the many points brought to light was the particularly interesting discussion of traffic conditions in Washington, D. C., by Messrs. Stark and

* Discussion of the paper by William J. Cox, Jun. Am. Soc. C. E., continued from October, 1927, *Proceedings*.

† Author's closure.

‡ Asst. Prof., Eng. Mechanics, Sheffield Scientific School, Yale Univ., New Haven, Conn.

§ Received by the Secretary, October 22, 1927.

Eldridge. Mr. Eldridge* is in a position to know the difficulties of traffic administration in Washington, but the writer is unable to agree with him that Washington streets are not well designed to carry the traffic. It is true that many of the streets in the city are narrow, but there is no reason why all the streets of a city, or even a majority of them, should be wide. Provided there is a sufficient network of arterial ways to carry through traffic, it is advantageous for several reasons that intermediate streets should not be too wide. Like all cities, designed before the invention of the automobile, Washington has an inadequate street system, but its streets are more nearly adequate for their traffic needs than those of any other large American city, and the distribution of business and population through the city is also unusually good.

Mr. Eldridge speaks of the hazard introduced by the convergence of a number of streets at certain points in Washington; as, for example, at Thomas and Scott Circles. The writer is not in possession of accident statistics for these two localities, and they may be more dangerous than they appear, but he has repeatedly watched the streams of traffic converging on them and diverging, and has been impressed with the smoothness and safety with which they move. He finds it difficult to believe that such circles add to the street hazards of Washington.

Mr. Eldridge calls attention to the decrease in automobile fatalities and accidents in Washington from 1923 to 1926, and suggests certain causes, denying that the improvement was brought about by the planning of the city, etc. This is true. From 1923 to 1926 there was no marked shift in the planning of the city. The writer's paper was based on 1922 conditions; and the improvement subsequent to that time, which no doubt is due to the causes enumerated by Mr. Eldridge, is outside the scope of the paper. The planning of the city and perhaps the superior administration of traffic, even before Mr. Eldridge took charge, in Washington made it possible for the improvement to which he refers to be based on hazard conditions that were apparently already surprisingly good.

Mr. Stark,† in vigorously dissenting from the views set forth by Mr. Eldridge as to the excellence of police regulation of traffic in Washington in recent times, makes a well-reasoned plea for a greater consideration of the pedestrian in traffic. He has convinced the writer of the error in stating that improved methods of handling traffic at intersections will have little effect on collisions between automobiles and pedestrians. He advances a sound argument that the effect, although indirect, is nevertheless present.

Mr. Williams‡ also stresses the indirect connection between traffic hazards and "the general public attitude toward law observance and accident prevention". The writer recognizes the weight of Mr. Williams' judgment in this matter and the very great importance of the relationship he mentions. The writer does not believe, however, that any differences in this attitude of the

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1270.
† *Loc. cit.*, p. 1272.
‡ *Loc. cit.*, October, 1927, Papers and Discussions, p. 2023.

public could exist sufficiently great to overbalance entirely the physical advantage of low population density. That is, he doubts if such an improvement in the attitude of citizens toward traffic rules in New York, coupled with such a retrogression in Indianapolis, could occur as to overbalance the differences in population congestion and place those cities on a par from an automobile-operation hazard standpoint.

Mr. Lewis* is apparently confusing the fatality rate per unit of population and the fatality rate per automobile. All the data to which he refers are based on the automobile-fatality rate per population unit, while the writer's paper was a discussion of the hazard per car—the hazard of operating an automobile. This was found to be $\frac{P}{M}$. As Professor Lyle points out,† the hazard per unit of population derived from this formula is $\frac{R}{M}$, which would seem to accord with Mr. Lewis' views.

Among those who discussed the paper, Mr. Kelcey‡ was the only one to express marked distrust of the use of automobile liability insurance rates to measure hazards. There were probably other readers who felt about this as Mr. Kelcey did, since there is a general disposition on the part of the public to suspect that insurance rates are established wholly competitively and are based on what the traffic will bear.

Mr. Kelcey's objections to the insurance rate as a measure were along three lines. He states that the proportion of taxicabs and commercial vehicles among the total number of cars insured in New York is much greater than in smaller cities, such as Indianapolis; that these public and commercial vehicles have much greater annual mileages than the average private passenger car, and, therefore, a greater annual hazard; and that, consequently, they distort the insurance rate of New York, making it unduly high in comparison with those of other cities where the proportion of the relatively safe private passenger cars insured is greater. Mr. Kelcey's argument would be sound, granting his premises, if the insurance rate were established, as he assumes, by totaling the premiums paid by cars of all classes, and dividing it by the total number of insured cars of all classes in the city. Insurance rates are not made this way, however, but separately for cars of each type, private passenger, public passenger, and commercial. As stated by the writer,§ the private passenger car rate, being by far the most important and the most accurately determined, is the one selected as a means of comparison. The variation from city to city of commercial and public passenger car rates does not differ greatly from that of private passenger car rates, and no material error is introduced by selecting the latter as a standard. The fact that the ratio of insured taxicabs to insured private passenger cars is greater in New York than elsewhere, if true, is without force as regards this discussion.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1281.

† *Loc. cit.*, p. 1269.

‡ *Loc. cit.*, p. 1285.

§ *Loc. cit.*, April, 1927, Papers and Discussions, p. 518.

A second question raised by Mr. Kelcey may be more pertinent. He suggests that, as insurance rates increase from cities of low hazard to cities of high hazard, the better class of risks drop out, leaving a higher proportion of "bad risks" in cities with high rates, thus continually raising the rates still higher. This is a view held by many people, but by no means universally; and among those who hold that there is such a tendency, opinion differs as to how marked and how important it is. Light will be shed on this question when the Massachusetts law, providing that every automobile must carry liability insurance, has been in effect longer. So far as the writer is able to learn, during the few months of its existence there has been no decided indication that the addition of the large number of those formerly uninsured to those already carrying insurance, either in Boston, with its high rates, or in small cities in the State, where rates are much lower, has either raised or lowered the loss ratio; that is, the newly insured cars, taken as a whole, seem to be neither worse nor better risks, in any decided degree, than those formerly insured.

The most serious objection Mr. Kelcey raises is that insurance rates are established, in considerable part, by business competition. In substantiation of this, he instances that the Detroit Automobile Club began in 1922 to sell insurance to its members at a lower rate than that of the stock insurance companies, and that within the next two years the stock companies' rate was reduced 30 per cent. He states that the Cities of Los Angeles, San Francisco, and Chicago "are understood" to have had a similar experience following the sale of automobile insurance by local automobile clubs to their members.

The loss experience sheets of the National Bureau of Casualty and Surety Underwriters show that on policies written in Detroit in 1922 the claim frequency was 4.0; that is, for every hundred policies written that year, 4.0 claims resulted. In 1923, this frequency dropped to 3.7, and, in 1924, to 3.0. Correspondingly, the insurance rate dropped 25% from 1922 to 1926. Thus, the rate trend followed the claim frequency trend, and no competitive influence of the automobile club is apparent.

In Los Angeles, claim frequency dropped from 4.4 in 1922 to 2.6 in 1924. The 1922 claim frequency was abnormally high, so that the improvement trend has not been so great as appears from these two figures alone. Since 1922, insurance rates have declined 20 per cent. Here, again, while a decline in rates has occurred, as Mr. Kelcey states, it is less than the decline in the frequency of claims. If the Automobile Club of Southern California has played a part in rate reduction, that part has consisted in its valuable hazard-reducing work in the traffic education field.

In San Francisco, despite the existence of the automobile club to which Mr. Kelcey refers, there has been a slight increase in automobile liability insurance rates since 1922. In 1926, they stood 5% above the 1922 level. There has been no sustained trend in the claim frequency. It was 4.0 in 1922 and, again, in 1923. In 1924, it dropped to 3.0, but, in 1925, rose to 3.7. The

average severity of accidents also appears to have been abnormally high in 1924 and 1925.

In Chicago, claim frequency dropped from 6.6 in 1922 to 6.5 in 1923 and 4.8 in 1924. The insurance rate declined 22% from 1922 to 1926. In none of these instances does the competition envisioned by Mr. Kelcey appear to have played a part in affecting the insurance rate level.

It is not the writer's view that public liability insurance rates make a perfect yard-stick for measuring hazard variations, but he does believe that they are decidedly the best measure available. He emphatically does not believe (and the figures support him) that insurance rates fluctuate competitively in the sense that gasoline prices do. Rates are not an immediate reflection of hazard fluctuations because they are damped (by the use of mathematical formulas, of country-wide application) to smooth out extreme peaks and hollows and, therefore, show only established, not accidental, trends. However, there is no question but that they are determined by such trends, rather than by business competition.

Mr. Kelcey apparently takes issue with the whole conclusion of the writer's paper that diffusion of population decreases the hazard of operating an automobile. He instances the infrequency of traffic accidents in the dense vehicular and pedestrian traffic at Fifth Avenue and 42d Street, New York City. He finds a contradiction in the writer's statements that, "diffusion of population leads to a decrease in the number of possible collisions per unit distance of travel and thereby reduces hazard" and "most automobile collisions with pedestrians occur in residential districts." Residential districts, he states, are areas in which the population is diffused; hence, the contradiction.

The fallacy in Mr. Kelcey's reasoning is that sometimes population is diffused in residential areas and sometimes it is not. Who would say that the Lower East Side of New York was not a residential area and who would say that it was an area of diffused population? The fact of the matter is that most pedestrian accidents occur in residential areas, and that, as a rule, these areas are the congested, tenement districts of the residential part of the city as a whole. The writer mentioned this in his "Conclusions".* Data since collected enable him now to illustrate the point numerically.

In New Haven, the highest class (single-family) residential area, where population is most diffused, is traversed by approximately 16 miles of streets. In this region, records of the Department of Motor Vehicles of Connecticut show that not a single pedestrian accident occurred during 1926. Approximately, 100 miles of streets traverse areas outside the central business district which are unrestricted for residential use. On this 100 miles of streets there were in excess of 200 pedestrian accidents in 1926.

Mr. Kelcey presents a diagram (Fig. 4),† based on data from the State Highway Department of Pennsylvania, which, to his mind, shows that the more diffused vehicular traffic becomes, the more hazardous automobile opera-

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 537.

† *Loc. cit.*, August, 1927, Papers and Discussions, p. 1288.

tion becomes. His data relate to rural, not urban, conditions, and it is questionable whether they do not indicate the reckless character of the traffic on the highways at 2:00 A. M., as much as anything else. He is correct, however, in thinking that decreasing the density of vehicular traffic may add to the hazard of automobile operation. The explanation is obvious. A very dense vehicular traffic must be controlled; and a controlled traffic is a safe traffic. A diffused vehicular traffic becomes an unregulated traffic, and often becomes a high-speed traffic, which is correspondingly dangerous.

The writer has not urged that vehicular traffic be diffused. He has urged that population be diffused, and that in passing through areas of diffused population, vehicular traffic be concentrated in arterial ways into streams of sufficient density that their continuity may bring safety.

Mr. Finch* expressed the idea much more effectively than the writer. "* * * in attacking the street accident problem as it now exists in any city," he states, "it is important to diffuse population and to concentrate [vehicular] traffic." In other words, spread out population until the smallest possible proportion is encountered in a given street, and then put the largest possible proportion of the vehicular traffic through that street, regulating it as may be necessary. As Mr. Finch points out, diffusion of population is often impracticable. Concentration of traffic is much less often impracticable; it is obviously good sense, but is often not done.

As an instance of this in the neighborhood of the writer's home, traffic on a "through" route traversing the city, has intentionally been routed over several parallel streets, although the volume of it is not so great that one street could not handle it all. As a result, sparse streams of out-of-town vehicular traffic pass through an apparently quiet residential section at fairly high speeds and, without warning, come to a cross street carrying a fairly high speed and fairly heavy, but often intermittent, city traffic. Some of the worst accident corners in the city are the result. The through traffic should be gathered into a condensed stream on one street, freeing the parallel residential streets of it; and this stream should be controlled by lights or otherwise where it crosses other traffic arteries. Almost every city can show examples of the same failure to appreciate this law of safe street use.

A number of other worth-while points were brought out in discussions. Mr. Cole† called attention to the fact that nine of the cities shown on the writer's Fig. 3‡ have been doing organized street safety work for some years, and that seven of these nine cities had "actual hazards" lower than their "calculated hazards." This is very significant as an indication of the possibility of modifying the "natural" hazard level.

Professor Kirby§ calls attention to the very great need of more adequate and better organized statistics relating to street and highway accidents. Mr.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1284.

† *Loc. cit.*, p. 1281.

‡ *Loc. cit.*, April, 1927, Papers and Discussions, p. 527.

§ *Loc. cit.*, September, 1927, Papers and Discussions, p. 1731.

Holleran* and Mr. Johannesson† deal interestingly with measures developed in their sphere of highway work to make highways safer. Space is lacking to comment adequately here on other points of interest.

Colonel Barber's‡ discussion emphasizes, in words that will bear repetition, the fundamental importance of more satisfactory planning of cities, as follows:

> "The modern conception of a city or metropolitan region contemplates the location of commercial, industrial, and other centers at appropriate widely distributed points, with housing and community facilities for employees in close proximity to each center. The working out of this conception should economize not only time and expense of travel, but should also tend to reduce the accident hazard."

As suggested, the street safety problem is tied up closely with the matter of the economical use of city streets; that is, with the basic economics of city life.

The problem of automobile accidents confronts the city as an existing condition. This fact has been often pointed out, and, unfortunately, people have already suffered enormously from the too great emphasis placed on it. In the last 20 years the population of most large cities in the United States has increased from 50 to 600, or more, per cent. There is reason to believe that such increases of urban population will continue to be typical. There is, therefore, every reason to give more constructive thought than has yet been generally given to the probability of such increases and to their street-traffic implications. By all means, cities should avail themselves of the expedients that Mr. Holleran's discussion enumerates and describes as palliatives rather than cures. However, with them they must give (and progressive cities are giving) increasing consideration to a more far-sighted view of future traffic needs and to the provision of the proper facilities to meet them.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1275.

† *Loc. cit.*, p. 1278.

‡ *Loc. cit.*, October, 1927, Papers and Discussions, p. 2024.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE EYE-BAR CABLE SUSPENSION BRIDGE AT FLORIANOPOLIS, BRAZIL

Discussion*

By Messrs. Leon S. Moisseiff, R. McC. Beanfield, and Charles F. Stowell.

Leon S. Moisseiff,† M. Am. Soc. C. E.—This paper is of much interest. It tells of the design of a new type of suspension bridge and how it has been successfully erected and put in use in a foreign country. It also tells of a novel material that has been used in bridge building. The engineers who have planned, designed, erected, and managed the enterprise and now report on it deserve much credit.

The form used for the stiffening truss of the Florianopolis Bridge has undergone a long evolution and has been discussed at various times in engineering offices and publications before it took its present shape. It may be well to sketch here briefly the development of the stiffened cable bridge.

Since the time when suspension bridges, stiffened by a railing, had been found to be too limber, it has been the aim of many engineers to stiffen the cable directly by making it serve either wholly or partly as a stiff member. The simplest form that this idea took was that of splitting the cable in two parallel chords placed vertically apart and bracing them to each other so as to form an inverted arch. This form has been realized in several smaller bridges and several decades ago it was proposed by Gustav Lindenthal, M. Am. Soc. C. E., for a bridge over the Hudson River. In 1874, the late James B. Eads, F. Am. Soc. C. E., patented a design of a braced cable in the form of an inverted three-hinged arch made of two bi-convex halves. The same idea found a simpler expression later in the bowstring type of which the Old Point Bridge, built at Pittsburgh, Pa., in 1878, by Hemberle, offered an excellent example. In 1875, Fidler, in England, proposed a modified type of bridge

* Discussion on the paper by D. B. Steinman and William G. Grove, Members, Am. Soc. C. E., continued from November, 1927, *Proceedings*.

† Cons. Engr., New York, N. Y.

consisting of two intersecting cables tied together by web bracing. Each cable was to take the form of an equilibrium curve with one half span uniformly loaded. The upper branches of these curves then become straight lines and constitute the upper chords of the stiffening truss. When the bridge is fully and uniformly loaded the equilibrium curve of the system will coincide with the neutral curve of the braced arch. The stress in the chords will always remain in tension. They can be built, therefore, as tension members exclusively. This, of course, offers a substantial advantage in economy.

It appears that Fidler's braced arch is but a special case of a general type of a suspension bridge formed of two curves intersecting at mid-span and braced to each other and possessing the characteristic mentioned, of tensile stresses in the chords exclusively. Résal, in France, published in 1893 a sketch, and, in the same year, Max am Ende, in England, published the general analysis, of this type of truss. Ende shows that by selecting a depth of truss at the quarter-points that is a function of the ratio of the live load to the dead load, two curves can always be adapted that will form the chords of two crescents hinged together. These curves will remain in tension under all positions of the specified intensity of live load. In other words, the axial stress, due to the weight of the bridge and the live load on it, will never be exceeded by the stress caused by the flexure of the truss.

In 1895, the speaker, under the guidance of William H. Burr, M. Am. Soc. C. E., selected as a thesis the then much discussed bridge over the Hudson River, with a span of 3 000 ft., and developed a design on this type. In 1910, the Pennsylvania Steel Company submitted several designs for the second Quebec Bridge. One of them was the design proposed by Mr. Lindenthal for a suspension bridge. It consisted in the center span of two intersecting curves forming two braced crescents hinged together at mid-span. The side spans consisted of braced segments formed of a lower curve and an upper straight line chord. In 1924, the McClintic-Marshall Company, among other designs for the Sydney Harbor Bridge, also proposed a 1 600-ft. span on practically the same lines.

German engineers have been always much interested in these forms of stiffened suspension bridges and since the Nineties many and various forms of this kind have been discussed in German engineering literature.

In 1903, Mr. Lindenthal, while Commissioner of Bridges of New York City, proposed an eye-bar bridge for what is at present the Manhattan Bridge. The main feature of his plan was a chain of eye-bars that would sustain the dead load of the bridge and form an equilibrium polygon. To the chain a truss was attached which consisted of a lower chord capable of resisting tension, and also a compression and a web system. The lower chord, beginning from the hinged support of the chain on the tower, descended to the roadway level at about the quarter-points and continued parallel with the roadway grade so that the middle half of the span was practically horizontal. The live load and temperature that would come on the bridge, would be jointly sustained by the chain and the lower chord forming an inverted braced arch.

In designing this bridge it soon became apparent that an additional wind chord would be required for the outer quarters of the span, or for about one-half the bridge. The speaker, who was in charge of the design, then arrived at the conclusion that the form now adopted for the Florianopolis Bridge would prove more suitable, as it does not require an additional wind chord and presents a more pleasing appearance.

In 1907, the speaker made a design for a bridge across the Kill van Kull between Staten Island, New York, and New Jersey, in which he proposed a stiffened suspension bridge of exactly the type adopted subsequently for the Florianopolis Bridge. An outline of its elevation and cross-section is shown in Fig. 42.

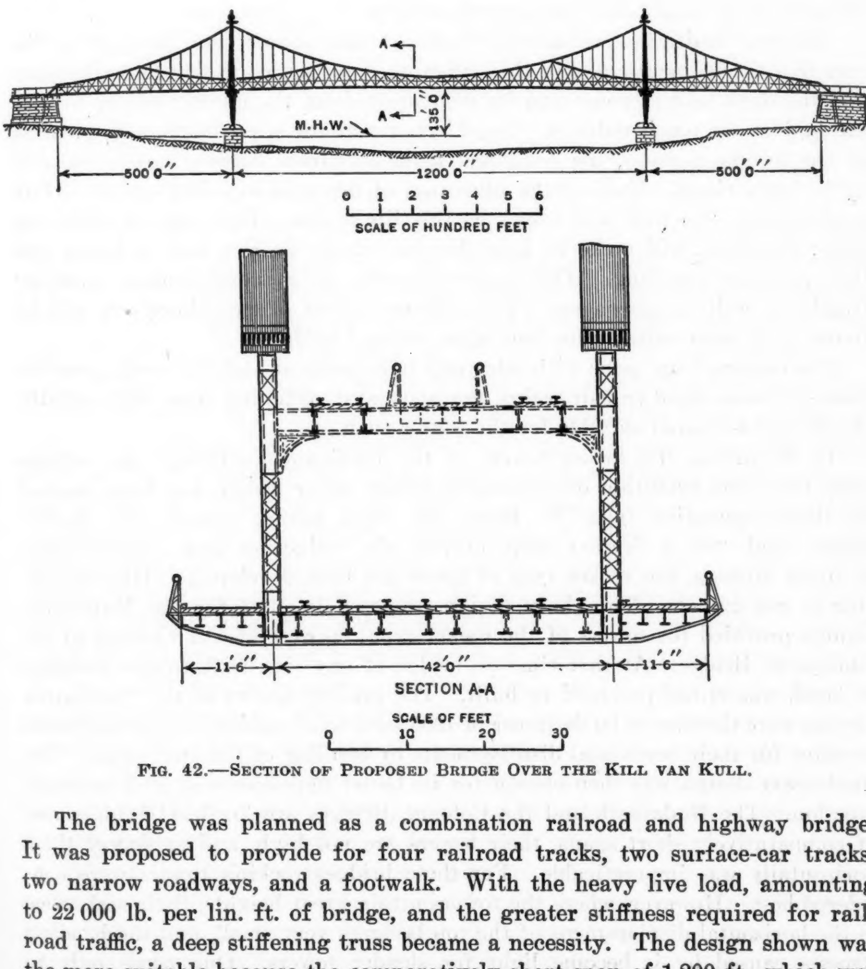

FIG. 42.—SECTION OF PROPOSED BRIDGE OVER THE KILL VAN KULL.

The bridge was planned as a combination railroad and highway bridge. It was proposed to provide for four railroad tracks, two surface-car tracks, two narrow roadways, and a footwalk. With the heavy live load, amounting to 22 000 lb. per lin. ft. of bridge, and the greater stiffness required for railroad traffic, a deep stiffening truss became a necessity. The design shown was the more suitable because the comparatively short span of 1 200 ft., which was slightly longer than that of the Florianopolis Bridge, lent itself to the economic use of nickel-steel eye-bars. At that time these bars were the strongest

structural steel bars available. The arrangement of the eye-bars is shown on the cross-section. The lower chord was utilized as the chord of the horizontal wind truss. For better appearance, the truss ends at the towers show the line of the upper chords continued.

The type of the stiffening truss of the Florianopolis Bridge, however, has its limitations. As the span grows longer the depth at the peaks of the truss becomes greater, with the result that the web members become excessively long and limber. This is still more accentuated by the fact that to obtain the most economy with this type of truss the versine of the chain should be relatively large and with it the depth of the truss at the peaks increases in proportion. Thus, for example, a span of 2 000 ft. would well result in a depth of truss at the peak points of more than 85 ft.

Another limitation which effects one of the reasons for the type is the fact that the moment curve of the stiffening truss begins to flatten as the span and the dead load increase and its depression from the quarter-points toward the middle are much reduced. Considering that the wind moment is greatest in the middle portion, the combined moment curves become nearly parallel to the lower chord. Much of the advantage of this type will thus be lost. Only a very heavy live load will overcome this limitation. This type of stiffening truss, therefore, will make its best showing where the live load is heavy and the span not too long. This points directly to railroad bridges carrying freight as well as passengers. The stiffened chain bridge, therefore, will be found to be most suitable for long-span railroad bridges.

For modern long spans with relatively light loads of highway traffic, surface cars, and even rapid transit trains, the suspended stiffening truss with parallel chords will be found suitable for the center span.

In discussing the rocker towers of the Florianopolis Bridge the authors state that "the evolution of suspension-bridge tower design has been marked by three successive types".* First, the rigid tower; second, the flexible tower; and "as a further step toward the reduction and simplification of tower stresses, the rocker type of tower has been developed." Historically, this is not correct. The chain design proposed in 1903 for the Manhattan Bridge provided for towers of the rocker type; so were also the towers of the Budapesth Bridge. At that time no bridge of any size, with towers designed to bend, was either proposed or built. The present towers of the Manhattan Bridge were the first to be designed in 1905 with fixed saddles and proportioned to allow for their horizontal displacements by bending of the steel posts. The fixed-tower design was then chosen for its better appearance as well as easier erection. The Budapesth and the Cologne Bridges are low-level bridges and of comparatively short spans, their towers are not high, and to deflect them horizontally was impracticable. For these bridges rocking towers were considered best. However, where the towers attain great heights, their resistance to the horizontal displacement of the top becomes very small, and the bending stresses caused by it become light for slender towers. One needs only to recollect that the unit stress for bending is inversely proportional to the

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 723.

square of the tower height. This is the guiding principle in the technical selection of the tower. Correctly designed and proportioned, a tall tower with a fixed base should hardly require more material than a rocker tower.

It may be of interest to state that the McClintic-Marshall Company, that is building the Detroit River Bridge with a span of 1 850 ft., has adopted towers of the fixed-base type.

R. McC. Beanfield,* Assoc. M. Am. Soc. C. E. (by letter).†—This paper presents much information of great value and considerable interest, especially from an economic standpoint, in the design and erection of long-span bridges. The novel features in the design and installation of this large structure have demonstrated some economical advantages that will doubtless have considerable influence in the planning of future long-span bridge projects.

The writer has had occasion to check the stresses in a structure containing several groups of eye-bar members. Extensometer readings were made with considerable care (with a Berry 8-in. machine) and temperature changes checked with a standard try-bar. Considerable variation in stress was found in the individual eye-bars that were grouped to form a member of the truss. While the cause of this variation in stress in each eye-bar is not definitely known, it is the writer's opinion that the pin clearance, minor differences between pin-hole distances that require extreme care in shop measurements, and secondary stresses and deformations in the eye-bar heads, were the underlying reasons.

It would be interesting to know if extensometer readings were taken on the eye-bar chains to ascertain if there was any great variation in stress in the individual eye-bars that were grouped to form a unit or link of the chain. By comparison, it would be of further interest to know if there is much variation in stress in the wire strands in some of the large suspension bridge cables.

One of the most important structural units in a suspension bridge is the tower castings supporting the cable. Maximum intensity of stresses in these castings should be comparatively low. In this connection, there may be some question as to the amount and intensity of the secondary stresses induced in the casting and connecting eye-bars due to fixity or lack of rotation in the pin joint. If the primary stresses in the eye-bars are not sufficient to overcome the frictional resistance of the pin, rotation will not occur, and the joint may be considered as fixed. According to experiments of Föppl, the coefficients of friction for steel pins on steel may vary from 0.25 to 0.29, which would indicate that the pin friction would permit stresses of about 50% in excess of the primary stress prior to rotation on the pin.

Data on the frictional resistance of pins are very meager. Any assumption made relative to the coefficient of pin friction is more or less a matter of guesswork which, in turn, reduces the reliability of secondary stress analysis relative to pin joints.

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* Structural and Mech. Engr., Los Angeles, Calif.

† Received by the Secretary, October 11, 1927.

The design of the rocker casting details at the base of the tower* presents some very interesting features that raise some questions, as follows:

(*a*) Security against displacement by earthquake shock, particularly by horizontal shear.

(*b*) The extreme high bearing stress on the contact surface of the rocker castings extending over a line of contact 45 in. long, on which 4 840 000 lb. must be supported.

The action of the horizontal component, or wave of an earthquake, sets up a violent movement of the earth's crust which, in turn, is transmitted to the lower supporting medium of the structure. The upper part of the structure, due to its inertia, tends to remain in its original position. Therefore, the hinged or flexible tower castings, together with the four 3-in. screwed dowels and columns, must resist violent successive forces (shears) before they can move the superstructure in the direction of the vibration.

Furthermore, the effect of the period of seismic vibrations on the induced stresses in the structure, combined with the period of elastic vibration of the structure itself, is a matter of considerable importance, especially in the tower frames, where nearly all the load of the suspended spans is supported on the top of the towers. This condition tends to intensify the oscillatory movement, thus increasing the destructive forces induced in the structure. For this reason, it would indicate that suspension bridges, in general, are not adaptable for locations known to have (active) seismic disturbances.

In general, more serious consideration should be given to the resistance to seismic forces in structures, particularly those projects involving millions of dollars in cost. It is a fact that there are no white spots on the earth that are free from seismic disturbances. There are ample records showing that the eastern slopes of the Western Hemisphere have had a fair number of very destructive earthquakes, such as a series of violent earthquakes in the St. Lawrence region in 1633, the Boston earthquake in 1755, the Missouri earthquake in 1811, and the Charleston earthquake in 1886.

Engineers on the Pacific Coast are giving serious consideration to the subject of seismic analysis of structures, probably more than ever, due to the influence of the recent Japanese earthquake. Also, since the Pacific Coast earthquake, the insurance rates have been unduly increased, the same applying alike to properly and poorly designed structures.

In the design and construction of the Carquinez Strait Bridge, in California, one of the largest cantilever type bridges in the world, resistance to seismic forces was considered by the installation of hydraulic buffers to absorb some of the shock and automatically unite the structure together, to increase its rigidity.

The cost of providing proper resistance to minimize damage from earthquake shocks is a very small percentage of the total cost of most structures; therefore, from an economic standpoint, it would be a wise policy.

Relative to the rocker bearing, which is virtually a linear contact 45 in. long, supporting about 4 800 000 lb., there must be considerable deformation

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 725, Fig. 7.

of the contact surfaces with a unit bearing stress of high intensity. It would be interesting and instructive to know the basis of design for this important detail.

CHARLES F. STOWELL,* M. AM. SOC. C. E. (by letter).†—The Florianopolis Bridge marks an epoch in suspension bridge construction and although it is of comparatively moderate dimensions for a bridge of that character, in design and construction it so typifies, in nearly all its elements, the most advanced ideas and research that it is likely to serve for a long time as an example to be followed by those who seek perfection in the building of such bridges. While many of the distinctive elements have been previously suggested, it remained for the designers of this bridge to put them into practical use for the first time; and the designers' clear and complete explanation of the steps and processes leading to their results make this paper a classic of its kind.

The one discordant note in the otherwise harmonious whole is the unfortunate predominance of the single element of cost, which prevented the use of wire instead of eye-bars for cables "by a small margin". The superiority of wire over eye-bars for cables is so great that a difference in cost "by a small margin" should not be a controlling factor in deciding between them.

In any long-span bridge of whatever type the dead load is always the chief part of the weight to be supported, and various devices are adopted to reduce this weight to its lowest terms. This is accomplished both by skillful designing and by the use of various kinds of alloy steel of greater strength than ordinary structural steel, thus permitting a smaller quantity to be used; or, as in this case, by special heat treatment of steel normally of less strength, to increase its strength and thus reduce the dead weight of the structure.

The cables of the Florianopolis Bridge are made of eye-bars, each cable composed of four bars 12 in. wide and approximately 41 ft. long and of a thickness varying from $1\frac{13}{16}$ to 2 in., proportionate to the varying maximum stress in the different panel lengths. The chemical composition of the steel used for these eye-bars is not given, so that its approximate normal strength before heat treatment cannot be estimated; but, after treatment, the thirteen bars used for full-sized tests gave an elastic limit (yield point) varying from 78 180 to 96 830 lb. per sq. in. and an ultimate strength of 115 970 to 137 900 lb. per sq. in., the specified requirements being a minimum of 75 000 lb. and 105 000 lb., respectively. The capacity of the machine on which full-sized tests were to be made, was limited to lengths of 40 ft., or less than the lengths generally to be used for the cables; so that of the twelve bars originally decided upon for such tests, only two were actually selected from those made and intended for use in the bridge, while the other ten were bars made expressly for testing purposes and not actually representative of those to be used in the bridge. It is contrary to the experience of human nature not to assume that, knowing that these ten bars were the ones made expressly for test purposes, extraordinary care would have been taken to make them in every particular as nearly perfect as was humanly possible; and yet one of them failed signally in that it did not give nearly the percentage of elongation expected and

* Civ. Engr., Albany, N. Y.

† Received by the Secretary, October 27, 1927.

specified; and so another test bar was made, which fortunately did fill all the requirements. How many of the other bars, which were not tested, might show similar results there is no possible means of determining. All the bars made, including the one that failed, were subjected to delicate and ingenious tests to determine their degree of hardness and the uniformity of their texture; but, unfortunately, no one can tell how much a bar is going to stretch without stretching it.

It may be interesting to consider what may be the result if even one of these were to have as low a percentage of elongation as the test bar that failed. This bar measured 12 by 2 in. and showed a percentage of elongation of 3.8 on a measured length of 18 ft., while the other ten bars tested showed an average percentage of 6.996, or, say, 7 for the same length. The bars of this size are mostly used in the back-stays (176 in all) and in each panel length there are four such bars, bearing a total maximum stress of 4 381 000 lb., or 45 635 lb. per sq. in., if all four pull equally. When placed under stress all these four bars must elongate to an equal amount and if any one of them is abnormal in respect of having a lower elongation ratio than the other three then that bar must bear more than its normal proportion of the total stress and the others will be correspondingly relieved. With one bar in a group of four having an elongation percentage of 3.8, while the percentage of the others is 7.0, means that, in order to stretch all of them to an equal amount, the abnormal bar will have to carry 35.2% of the total load and each of the others 21.6% thereof; that is, for a maximum total stress of 4 381 000 lb., the abnormal bar will carry 1 542 112 lb., or 64 250 lb. per sq. in., while the three normal bars carry 2 838 888 lb., or 39 429 lb. per sq. in., instead of the intended maximum of 45 635 lb. per sq. in. which all were designed to carry.

This, of course, is pure hypothesis and is only offered as an illustration of the uncertainty necessarily incidental to the use of chains of this type and against which there is no positive insurance.

With every kind of forged or riveted structural members, full-sized tests are destructive to the members tested and, at best, knowledge of the actual strength of the members used in the structure is based inferentially on the behavior of similar members tested to destruction; but with cable wire two full-sized tests on every individual wire in the cable can be made; and these tests, moreover, are always made on parts of the wire known from experience to be the poorest, namely, near the ends. The process of wire-drawing itself is a good criterion of quality and uniformity. Occasionally, but not very often, a wire will break in drawing, which indicates some undiscovered flaw or inequality in the metal, and, of course, causes the rejection of that particular wire. There is no kind of steel construction about which there is such absolute and accurate knowledge of qualities and strength as modern wire bridge cable.

Another advantage of wire over eye-bar cables is the very much less weight of metal used for splicing purposes. In the Florianopolis Bridge the cable bars are connected at intervals of about 41 ft. and the amount of metal used for connection purposes only, consisting of eye-bar heads, pins, nuts, etc., is obviously large, but the given data are not sufficient to estimate it accurately. In a wire cable the wires are ordinarily about 3 300 ft. long and weigh approx-

imately 330 lb., each wire being spliced to the next one by a sleeve-nut. In the Manhattan Bridge these sleeve-nuts are $1\frac{3}{8}$ in. long and weigh about $\frac{1}{2}$ oz. On the Delaware River Bridge they are 2 in. long and weigh about $\frac{3}{4}$ oz.

In an eye-bar cable, moreover, there are many places accessible to moisture, which cannot be reached by a paint brush or even by a spray, such as the spaces between eye-bar heads and, in the case of elongated eyes, as in the Florianopolis Bridge, the vacant part of the eye. In a wire cable, the entire interior is saturated with a rust-resisting mixture, tightly squeezed under powerful pressure, tightly wrapped with galvanized wire, and, finally, painted, making every part of the cable as nearly proof against rust as it possibly can be.

The strength of ordinary cable wire, moreover, is such that it may safely be used for a maximum unit stress of 80 000 to 100 000 lb. per sq. in., as against 46 500 lb. for heat-treated eye-bars, thus effecting a further saving in the important item of dead weight.

There is, apparently, among some steel manufacturers a fascination for secrecy in parts of their work, which is amusing even if rather silly. In olden times metal workers ascribed their success in producing some particular results to conjuring, and apparently the idea or something like it seems still to persist. One of the largest manufacturers of wire used to, and perhaps still pretends to, claim that there is a mystery about the bath of molten lead through which the wire is drawn for the purpose of so-called "galvanizing", and the receptacle for this bath is carefully locked and guarded throughout the process. A large manufacturer of steel plates for rust-resisting floor-treads declines to furnish any analysis of his product on the ground that it is a secret that he prefers not to make public, apparently oblivious to the fact that any one who wants such an analysis can very easily make it. As a matter of fact, these plates are made of an ordinary grade of steel with the addition of as much copper as can be used without making it "red short". It is a good mixture and well adapted for its purpose, but the pretense of secrecy about its composition is amusing. In the case of the eye-bars for the Florianopolis Bridge the manufacturers adopted a policy of secrecy and refused to permit any investigation of their proceedings; whereupon the engineers wisely declined any responsibility for such bars. Heat treatment of steel for the purpose of increasing its strength is not a new idea, but its application on a considerable scale has not been practiced until recently, and although the manufacturers of these bars deemed it necessary to clothe their operations in a smoke-screen of mystery there is no reason to believe that any skillful eye-bar maker could not duplicate their results after very little experimenting and without any pretense of conjuring. Apparently, however, the makers of these bars did not foresee that their secret operations would necessarily lead to the result that engineers in charge of work would decline responsibility for any manufactured article produced by pretended secret processes. It is gratifying to learn that the manufacturers of the eye-bars have accordingly decided to discard the element of secrecy, under the somewhat tenuous pretext that the process is now so far perfected that resort to secret operation is no longer necessary.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

URBAN AND INTERURBAN BUSES

Discussion*

By Anson Marston, M. Am. Soc. C. E.

Anson Marston,† M. Am. Soc. C. E.—The speaker began his professional life in railway location and construction, and his memory extends back to a period when "wild cat" railways were still being promoted and sometimes constructed. Those who have never worked on a "wild cat" railway, uncertain of ever getting their pay, have missed a wonderful experience.

The railway system of the United States was not laid out originally as a National system, carefully planned. It was developed by a method of "trial and error". Roads were built in numerous instances because localities could be found that would offer attractive bonuses. Later, many such lines, or parts of lines, have had to be abandoned.

The speaker has been a member of the State Highway Commission of Iowa for 22 years and wishes to discuss motor truck and bus transportation from the standpoint of the State Highway Commission. The highways of the United States are passing through a period similar in some respects to the early stage of railway development, referred to previously. No sufficiently scientific study has yet been made of the proper design of an adequate National highway transportation system for the United States. Development of highways and of highway transportation has been, and still is, proceeding to a considerable extent by a method of trial and error. The speaker does not feel that such a method is wholly objectionable. Scientific research should, in general, proceed by a method that permits variation of technique, and even of immediate objective, from time to time, as the facts developed in the investigation may indicate to be wise.

From the standpoint of State highway departments, the present situation as to the development of motor-truck and bus transportation on the highways

* Discussion on the paper by Britton I. Budd, Esq., continued from August, 1927, *Proceedings*.

† Dean of Eng., Iowa State Coll., Ames, Iowa.

is unsatisfactory. Such traffic has developed unsought, and its present chaotic condition constitutes an almost impossible situation, very much as if the electric or steam railways had to build bridges and roadbeds without knowing the kind of rolling stock or the amount of traffic to be anticipated. Even in States that have adequate legal restrictions on motor-truck and bus transportation, there is in most cases little effective enforcement of the law.

Nevertheless, the speaker adheres to the general belief that motor-truck and bus transportation is here to stay. State highway departments are intensely interested in the early development of adequate regulatory measures. Immense sums of money are being spent on the construction of permanent highways. The interests of the public require the early adoption of measures by which these costly structures can be protected against excessive damage by what, after all, constitutes only a small percentage of total highway traffic.

A study of transportation needs in Iowa indicates that there are important sections that are likely to be entirely dependent on truck and motor-bus transportation for passenger and freight traffic. Some railway branch lines have already been abandoned. In fact, in more than one instance, a State highway has been built in part upon the abandoned roadbed of a former railroad. The speaker believes that the railroads are finding it unprofitable to operate several other branch lines not yet abandoned. The longest interurban railway in Iowa is already operating motor buses for passenger transportation between different points on its line. The speaker believes that this practice is likely to extend to the steam railways in some instances.

Nevertheless, there may be a reaction in favor of street cars and railway cars, showing that they are preferred from the standpoint of convenience and safety. It may be that the present preference for motor-bus transportation is to some extent a temporary fad.

The speaker has had the opportunity, as a bystander, to observe two interesting instances of experience with motor-bus transportation as a substitute for street railway service. The first was in the case of a town of 10 000 population, where for a time a motor-bus system operated in competition with the street railways. The bus operation continued until the buses were worn out, when the company went into bankruptcy. A serious accident occurred shortly before the bankruptcy in which, by a gasoline explosion during the filling of the tank, some people were injured, and one injured passenger died. The bus company was so irresponsible that no damage suit was filed against it, but suit was brought against the gasoline filling station. After the bankruptcy of this system the street railway company secured a franchise and is operating a bus system of its own, and the combination is giving satisfaction to date.

The second instance was in the case of the City of Des Moines, which had a population of 141 441 in 1925. On account of a controversy between the electric railway company and the citizens over a franchise, the street railway entirely suspended operation, and the city was left entirely dependent on motor buses. After a comparatively short period of this experience, the franchise was voted as demanded by the street car company.

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PAPERS AND DISCUSSIONS

TIDES AND THEIR ENGINEERING ASPECTS

Discussion*

By Messrs. Victor Gelineau, W. M. Black, and Francisco J. Gaston.

Victor Gelineau,† M. Am. Soc. C. E.—This paper is a valuable and illuminating treatise on a difficult but very real and practical subject. It contains a vast amount of information for engineers whose work extends to tidal waters and who are frequently required to determine the elevations of high or low water. The evaluation of tidal ranges and elevations is a vital requirement in land drainage or reclamation works and in the construction of marine works of every kind, such as navigation canals or channels, basins, wharves, sea-walls and jetties.

The author has treated a subject that has assumed great importance in the State of New Jersey; and one that has taken an advanced position in the administration of its waterways and ocean frontage. Among the activities to which this statement applies are: (*a*) The sale and lease of lands flowed by tide water, the proceeds from which, amounting to many millions of dollars, are by constitutional provision dedicated to the support of free public schools; (*b*) the construction of coast-protection structures on which the State has expended large sums; (*c*) the dredging, wherever necessary, in the improvement of an inland waterway route, approximately 120 miles in length, through the coastal lagoon system from Cape May to the Manasquan River; and, (*d*) the opening or closing of ocean inlets.

In the sale or lease of riparian lands flowed by tide water, one of the first requirements is the location of the mean high-water line (low-water line in some States) that forms the boundary between State owned and privately owned property. Since the right to accretion depends on riparian ownership, that is, adjacency to the limiting line of tide water recognized by the laws of the particular State, property rights of the utmost importance

* Discussion on the paper by G. T. Rude, M. Am. Soc. C. E., continued from November, 1927, *Proceedings*.

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depend on an accurate determination of the shore line; that is, the mean high-water line or other tidal plane recognized by the law. The tide line then is a natural boundary, the position of which determines valuable property rights.

In the design and construction of wharves, bulkheads, jetties, or other water-front structures, the determination of low and high-water elevation constitutes one of the first requirements of design. In works of this nature it is the practice to place wales or stringers and other braces at or near low-water level. Serious disputes may arise over the determination of the low-water plane, especially because the contractor has only a short period during low-water slack tide in which to perform the operations at or near the low-water mark.

In the improvement or extension of waterway channels and the construction of navigation canals, the determination of the plane of reference is the first duty of the engineer. It is the object of improvements of waterways for navigation to provide a minimum depth of water at a given stage of tide, usually local mean low water, to be available when the project is completed.

The elevation of local mean low tide that is established before dredging a channel of relatively large cross-section through a tidal lagoon region, will differ in greater or less degree from the elevation of the mean low-water plane determined after dredging. The engineer must estimate in advance of dredging just how much this disparity will amount to because the enabling legislation or other regulation prescribes the minimum depth to be obtained in the channel after dredging.

This shifting of the low-water and high-water planes, which follows and is a result of the dredging operations, depends on various hydraulic factors. The disparity between the old and the new planes amounts, in some places, to a considerable proportion of the entire mean tidal range. If the dredged channel cuts through reaches of congestion, such as shoals and tortuous minor channels, relatively extensive changes in the tidal regimen of the inland waterway are to be expected.

The establishment of mean low water or other tidal datum planes is effected usually by one of several different methods, as:

(*a*) A relatively short series of comparative readings of tidal swings at the unknown station simultaneous with readings at an established primary tide gauge station. This method, which disregards any correction factor, requires for applicability a close similarity of tidal regimen at the two stations of comparison.

(*b*) Simultaneous comparisons during a relatively short period of time at the new or unknown station and at the primary or reference station, utilizing an adjustment factor to correct for dissimilarity in the tidal regimen at the two stations of comparison; this is virtually an adaption of Method (*a*).

(*c*) Continuous independent readings of the tidal oscillations for a sufficiently long period of time.

The choice of methods will depend on many conditions, such as the degree of accuracy to be obtained and the time and equipment available. The chief

defect of the method of comparative readings is the fact that the effects of strong winds which modify the tidal swings may not be felt with equal force at the two stations of comparison.

All these methods have been used in the surveys by the New Jersey Board of Commerce and Navigation. The experience gained in these operations has convinced the writer that whenever practicable the third method, namely, that of independent establishment of tidal planes, should be resorted to frequently and should be applied for each section of waterway that differs appreciably in tidal range from the adjacent station.

Experience in New Jersey has demonstrated that continuous readings of the tidal range for one lunar month may be, and usually are, utterly inadequate for the independent establishment of the mean high or low-water plane at any station. The error in the result will probably exceed the limits permissible in projects to dredge inland waterway channels.

W. M. Black,* M. Am. Soc. C. E. (by letter).†—This subject is one of great importance to all engineers engaged in any work affected by tidal action. The paper is opportune and valuable. As stated by the author, the tides of the seas have been the subject of investigation for centuries, and their action is now well known. Tidal action in estuaries and in tidal rivers is not so well understood. The best discussion of tidal action in estuaries, known to the writer, is that of L. Bonnet.‡

Due to Commander Rude's well-known reputation as an expert in tidal movements, and his position in the U. S. Coast and Geodetic Survey, his paper must be considered authoritative. However, it contains a few statements which the writer, from the standpoint of an engineer who has had to deal practically with tidal action in estuaries and tidal rivers, deems likely to be misleading.

The theory followed by Commander Rude divides tidal waves into two classes, namely, progressive waves and stationary waves.§ This classification is not universally accepted. The classification of waves in fluid bodies as given by Bonnet seems to lend itself better to the solution of the problems of tidal action in confined tidal waterways, than that of Harris.‖

According to Bonnet, wave movements may be divided into two great classes: Waves of oscillation and waves of translation. Waves of oscillation are produced when a vertical force, such as the attractions of the heavenly bodies, or the fall of a heavy body, acts on a body of water, producing a momentary elevation or depression of the surface over a limited area. These waves have the characteristics of the progressive tidal waves described by the author.¶

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† Received by the Secretary, October 17, 1927.

‡ "Contribution à l'Etude Théorique des fleuves à Marée et Application aux Rivières à Marée du Bassin de l'Escaut Maritime."

§ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, pp. 1071 and 1081.

‖ "Manual of Tides."

¶ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1081.

Waves of oscillation may be periodic or ordinary. Periodic waves follow each other as ordinary waves, but the periodicity of movement follows periodicity of action, each action causing the formation of a wave. Ordinary waves are those produced by a force which acts once, or at irregular intervals.

The tidal wave of the ocean is a periodic wave of oscillation. Waves formed by a stone falling into water are ordinary waves of oscillation. These waves may be divided into two other classes: Those which seem to travel along the surface of the water, named in French, "ondes houlouses", which may be translated into English as "swells"; and those which seem to remain in one place, "ondes clapoteuses", for which the word "chops" seems the best English equivalent. The "chop" is formed in liquid masses confined by walls. Under such conditions a "swell" is reflected from the walls, with the angle of incidence approximately equal to the angle of departure and with the direction of propagation reversed. It superposes itself on the direct wave. The "swell" is the natural movement in open and deep seas when perturbing influences are negligible in amount, while the "chop" is formed in confined waters, such as rivers and small lakes.

If a force acts horizontally on a liquid mass, a wave of translation is formed. This is propagated above or below the original water surface. In the first case, it is termed positive; in the second, negative.

Positive waves of translation were first investigated by Scott Russell.* They are formed in a channel by a rapid elevation of the water surface, as manifested down stream by the opening of a movable dam, or in advance of a moving canal-boat. The forward movement of a flood crest in the Mississippi, in advance of the waters causing the flood, is another example. Negative waves are produced by the rapid creation of a depression in a liquid mass, as in the upper pool of a canalized river by opening a movable dam.

The distinction between waves of oscillation and waves of translation is of importance to engineers. In the former class, slack-water, which marks the change of current direction, occurs nearly midway between high and low water; in the latter class, it occurs at or close to high and low water. The tidal waves of estuaries and tidal rivers of the United States at the entrance are generally waves of oscillation which gradually change into waves of translation in their course toward the limit of tidal action. If it is desired to control only an ebb current at any point, and this current is produced at that point by a wave of oscillation, it is evident that the training wall need only be built to about the level of mid-tide, while if the tidal wave at that point is a wave of translation, the work must be built to high-water level.

A wave is formed through the expenditure of energy from some source, and once formed, it must continue until its energy is exhausted. The energy of a wave is measured by the work it can do in coming to rest. The total energy comprises the potential energy due to height and the kinetic energy, or half the living force, due to the velocities of its fluid molecules. The

* "Wave Action in Relation to Engineering Structures," Gaillard, p. 23.

energy of a wave of oscillation in deep water has been shown to be approximately equal to the product of the liquid mass contained in the orbit of the surface molecules by the square of the velocity of propagation of the wave.

In open and deep waters, when resistances due to friction and obstructions may be disregarded, the theoretical velocity of wave propagation approaches closely to the true velocity. This theoretical velocity is given by the formula, $v = \sqrt{g\,h}$, in which, v equals the velocity of propagation; g is the acceleration due to gravity (32.17 ft. per sec.); and h is the depth, in feet. In shallow and crooked channels this formula does not give results sufficiently accurate to be of great value to an engineer.

When a wave of oscillation approaches a coast obliquely from the open sea, the end of the wave nearest the shore is retarded by the shoaling waters, while the other end continues to advance with its original velocity. Waves thus seem to swing toward the coast line. When such a wave is propagated through a broad channel, deep in the middle, and with shoal areas along the sides, the same phenomenon is observed.

When a wave enters a widely opened mouth of a bay normally, the two extremities nearest the shores, where the water is shoal, advance more slowly than the central portion. The wave then extends into a fan-shape which becomes more marked as it progresses into the bay. In a funnel-shaped bay, with its mouth seaward and with decreasing depths landward, waves entering are shortened and become higher and higher. On the contrary, when waves enter a broad, shoal bay through a narrow channel, they become longer and diminish in height.

In estuaries and tidal rivers, the tidal wave is due to the ocean tidal wave at the mouth. After entrance, the wave is acted on by the forces which produced the ocean wave, very slightly, if at all. The energy of an estuary, or river, tidal wave at the entrance is that of the incoming ocean wave. After entrance the energy gradually is exhausted by the work of friction on the bottom, and by the resistances caused by bends or obstructions in the channel. Since this tidal energy is the source of the currents by which the navigable channels are maintained, it is the task of an engineer engaged in the formation and maintenance of such channels to see that the works constructed by him are such as will conserve this tidal energy as much as possible, for the benefit of the parts of the waterway lying beyond; and, further, to see that he does not make undue demands on the energy available by forming channels with dimensions too great for it to maintain, or by obstructing, unduly, the free passage of the wave.

Commander Rude mentions* the increase of the range (and, therefore, of the energy) of the tidal wave of New York Harbor due to the opening of the Ambrose Channel across the outer bar. He shows also the effect on the Delaware River tidal wave of the work done in the channel of the Delaware River below Philadelphia, but above the entrance channels to the river, as contrasted with the effect at Trenton, N. J.* A similar change in the tidal range

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1105.

of the Hudson at Troy, N. Y., resulted from the enlargement of the channel between Troy and Hudson, N. Y. In 1876, the tidal range at Troy was 0.8 ft., with the mean low-water plane 3.43 ft. above the Barge Canal datum. In 1910, the range was 2.06 ft. with the mean low-water plane 2.2 ft. above the same datum.

The writer is in hearty agreement with the statement* that,

"A fertile field remains open for the engineer in a study of the changes in the tidal regimen in an estuary or in a river resulting from changes in topography and depth in harbor and river improvements, in order that dependable quantitative predictions may be made for proposed projects."

Commander Rude's paper, as well as other papers by officials of the U. S. Coast and Geodetic Survey, give clear statements of current action in estuaries under the physical conditions which existed when the investigations were made, and when data are available, of tidal changes following changes in channel dimensions; but they do not attempt to state what changes would occur in the velocity of wave propagation, in the tidal range, and in the currents produced by the tidal wave action if extensive changes should be made in the physical conditions of the channel by works of river and harbor improvement. It is of maximum interest to engineers engaged in such works to know what such changes would be. In so far as is known to the writer, M. Bonnet is the only engineer who has gone into this subject extensively. He has been able to deduce formulas by which can be predicted the effect of physical changes in channel dimensions of the Scheldt River on the tidal action in that river.

In restricted tidal waterways, the tidal wave is always propagated at a velocity which is greater than the velocity of the currents produced by the wave. In such waterways the character of the wave frequently changes in its course from the entrance to the upper limit of tidal action. In the Hudson River, for example, the tidal wave when passing the Battery, in New York City, has the characteristics of a wave of oscillation, with the ebb slack-water occurring about midway between low and high water. As the wave progresses up the Hudson, the times of slack-water approach more and more closely to the times of high and low water, until above the City of Hudson, the times of slack-water coincide with the times of high and low water, a flood current beginning at low water. The wave there has the characteristics of the progressive type (Bonnet's nomenclature). The same is true in the East River where the wave of oscillation entering at the Battery becomes transformed into a progressive wave between the Brooklyn Bridge and Blackwell's Island.

Bonnet states with truth that it is not easy to define, offhand, the exact nature of a river tidal wave, "for it is one of the most complex of the phenomena of river hydraulics." He gives the following theory to account for the formation of river tidal currents.†

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* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1104.

† "Fleuves à Marées," Bonnet, Pt. II, Chapter II.

At the instant of low water at the mouth the portion of the wave in advance which is in the river, or the entire wave in advance as is the case in the Amazon and St. Lawrence, has a certain discharge toward the sea which is a function of the resistances of the watercourse and of the volume of flow from above.

The ebb current thus formed is increased, as the tide rises, by the volume of water left behind by the portion of the new wave advancing in the river; at the same time, this new wave itself tends to produce an up-stream current, which tendency is continuously augmented and finally neutralizes the counter down-stream current completely. At this instant, low-water slack occurs. When the manner in which this slack is produced is considered, it is seen that it must always take place when the tide is rising, even when there is no discharge from up stream. The length of the period which elapses between the time of low water and the time of ebb slack will depend on the resistances offered by the river channel and on the volume of the fresh-water discharge. After the time of ebb (low-water) slack, the counter (down-stream) current is less powerful than the current of the tidal wave, so that the resulting velocity is a flood (up-stream) velocity.

This phase of the tidal action persists until the volume of water left behind by the tidal wave in a unit of time, added to the fresh-water discharge, becomes again equal to the up-stream flow produced by the tidal wave. At this instant there is produced a new slack-water, known as flood (high-water) slack. The position of this slack with respect to high water, is no more definitely fixed than is that of low-water slack with respect to low water; but observations made on the Scheldt and its tidal tributaries show that high-water slack is always found after high water, except in the upper reaches of the tidal rivers, where, under the influence of the fresh-water discharge, the flood slack precedes high water. Close to the limit of tidal action a reach is found where the flood slack coincides with the ebb slack. Above this point, nothing but an ebb current is found, and the rise of tide is caused entirely by water from up stream.

These conclusions are the same as those reached by Van Brabant in his note on the changes of current in the river tidal wave.* After flood slack, the sections of the tidal wave growing smaller and smaller, the flow of the counter current is greater than that of the tidal wave; so that an ebb current prevails in the river. This phase of the tide persists until the subsequent ebb slack, after which a new cycle begins. The existence of a counter current explains the abnormal reduction of the velocity of propagation of the river wave and the rising of the mean level of the river in an up-stream direction.

To summarize, it can be stated that the river tide is a resultant of the combined action of a tidal wave, which goes up the course of the river, and of a return current, which is formed as the effect of the work of the river-channel resistances and of the fresh-water discharge.

* "Fleuves à Marées," Bonnet, Pt. III, Chapter II.

M. Bonnet makes a mathematical analysis of the effects of the frictional resistances encountered on the energy of a unit length of a tidal wave advancing up the river, and deduces formulas by which the tidal action in the Scheldt can be found. These formulas contain certain constants, the values of which vary for different streams, being dependent on the physical features of the river beds, which values must be determined by observation for each tidal stream.

It is to be hoped that future investigation will show the relation between these constants and the physical conditions of the tidal channel clearly enough to enable formulas to be deduced which will have a general application.

With reference to the practical application of the formulas to the problems of the improvement of any tidal stream, the writer agrees with M. Bonnet that:*

"When a project for the improvement of a tidal river is being prepared, the first task to be accomplished is the study of a mean tide of the river by means of direct observations and the computation of volumes. This preliminary study will give accurate data on the high tide cross-sections, the mean depths at mid-tide, the tidal ranges, the volumes of the flood currents, the fresh-water discharge, the mean level of the river, the velocities of the propagation of the tidal wave, the velocities of the flood and ebb currents, the periods of tidal rise and fall and of ebb and flood flow; in a word, on all of the important elements which make up the regimen of the river.

"When this preliminary study shall have been made, a possible longitudinal profile of mean depths at low tide can be determined. This profile should be selected so that there may be assured a good tidal propagation, the extinction of the river wave at the limit of tidal action in the river, the safe discharge of freshets, and the navigable capacity desired. Generally, the selection of the series of proper mid-tide mean depths will be a simple matter, since the river in its natural state gives sufficiently exact indications in its good and bad reaches of the depths to be preserved or to be modified."

From the foregoing, one of the river tide phenomena, which for a long time puzzled the writer, can be understood readily; that is, how the changes of direction of the tidal currents occur. In the Hudson, the tidal wave is more than 100 miles long, and the waves follow each other at intervals of about 12 hours, but the ebb and flood currents have a movement of only 10 to 15 miles before they change. The description of tidal wave action previously given shows that the ebb and flood slack waters, which mark the times of the changes in current direction, occur at definite periods after low and high water. As the wave progresses up a river, the slack-water phases arrive successively from point to point, so that an ebb or a flood current starts at successive points. The slack-water phases succeed each other at intervals of from 4 to 7 hours, the periods of ebb flow being the longer. At the end of each period, at each point along the river, the current direction is reversed. Since the current velocities are usually less than 2.5 miles per hour, the total distance through which a float will be moved by the current is from 10 to 15 miles. Such a float, started near the head of tidal action, will pass down the

* "Fleuves à Marées," Bonnet, Pt. IV, Chapter I.

river with the first ebb; will return to within a few miles of its starting point with the following flood; will then again start down the river; and thus gradually, after many tides, will reach the sea.

While, in a tidal waterway, for long reaches the longitudinal profile of mid-tide levels is approximately a straight line, the longitudinal profiles of the local mean high and mean low-water levels are irregular curves, as shown in Fig. 22.* This shows the necessity for establishing tide gauges as reference points at comparatively short intervals along a waterway, for the reduction of soundings in hydrographic surveying. Otherwise, true mean low-water depths cannot be found. Another phenomenon frequently found in such waterways is that at certain points, during certain phases of the tide, the surface slope of the water is up, measured in the direction of current flow; or, in other words, the water seems to flow up hill.

Referring again to Fig. 22, in his study of the Hudson River, the writer found that the longitudinal profile of mid-tide levels was formed approximately by three straight lines. Starting from the Battery, at New York, the profile had but a slight inclination upward until above Spuyten Duyvil, where a break occurred and the inclination from the horizontal became slightly greater. Thence, to a point above Hudson, N. Y., on the profile was a straight line. Another break occurred at this point, and thence to Albany, N. Y., the inclination up from the horizontal was markedly increased. In the writer's judgment, the break above Hudson marks the change from estuary to tidal river conditions.

In the course of his duties the writer has had occasion to study the data available on the tidal action and the resulting currents in the Cape Cod Canal and to make a careful survey and study of the complicated tidal conditions at Hell Gate in the East River, New York. An analysis of conditions in the Cape Cod Canal as they existed shortly after the Canal had been opened, has been published.† The surveying methods used at Hell Gate, and the conclusions of the writer have also been published.‡

The writer's experience with respect to currents formed in a narrow strait which connects two larger bodies of tidal water, is in agreement with that of Colonel Brown, as given in his discussion.§ Commander Rude's statement that, "The movement of the water is largely hydraulic in character," is incorrect if it means that the laws governing the ordinary flow of water in open channels are applicable in making predictions as to the effect of changes which will result in the currents of a channel connecting two bodies of tidal water, when the length, curvature, and dimensions of that channel are changed. From the standpoint of an engineer, the term used by Commander Rude is so broad as to be misleading. The hydraulic character of the movement through Hell Gate is the effect of tidal actions. To be able to modify this movement in the interest of navigation, an engineer must seek the cause.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1106.

† "The Cape Cod Canal," by William Barclay Parsons, Hon. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1.

‡ H. R. Doc. 188, 63d Cong., 1st Sess., "Survey of East River and Little Hell Gate, New York, and Resurvey of Hell Gate"; also, *The Military Engineer*, Vol. XVIII, July–Aug., 1926, p. 266, "Needed Harbor Improvement."

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 2058.

The writer is convinced that the conditions at Hell Gate are the result of the combined action there of two tidal waves, one reaching that point through Long Island Sound and one through Lower and Upper New York Bays and the East River. The rates of propagation and the progress of each wave can be traced clearly from the Tide Tables of the U. S. Coast and Geodetic Survey as far as Hunt's Point on the north and the north end of Blackwell's Island on the south. Between these two points the evidence given in the Tide Tables is not so clear. From these as well as from the current tables published by the U. S. Coast and Geodetic Survey, it is seen that the rates of propagation of the tidal waves are greater than the velocities of the tidal currents produced by the waves, until Hell Gate is attained. The onward movements of the waves do not cease when these points are reached, since the tidal curves show that their energies had not then become exhausted. Tidal curves taken between Hunt's Point and the north end of Blackwell's Island show clearly the superposition of the waves, and that they do meet in, and north of, Hell Gate. Other evidences of this action could be named.*

This question is of more than passing importance to an engineer. If the currents in Hell Gate are purely hydraulic in character, it would follow that an increase in the dimensions of the cross-sections of the connecting channels in Hell Gate would reduce friction and cause increased velocities. If, on the other hand, wave action does exist between the limits named, such action would cause, first of all, a change of "head" between the two points, by permitting a freer propagation of the waves, with a corresponding effect on the velocity of the currents.

An analysis of the tidal wave movements in the East River, more detailed than has yet been made, will be necessary before it is possible to predict with even approximate accuracy how, and to what extent, an increase in the dimensions and curvatures of the three channels at Hell Gate, and of the two channels to the south will change the rate and amount of tidal wave propagation, the surface slopes, and the currents in that vicinity.

Bonnet's theory of the action of a river tidal wave, and his method of analysis of tidal conditions, if followed, would throw great light on the problem of the reduction of the rapid currents in Hell Gate, a problem of primary importance for New York and the nation. A similar necessity exists for the further study of the tidal action in the other tidal waterways of the United States.

Francisco J. Gascon,† M. Am. Soc. C. E. (by letter).‡—This paper is most interesting and valuable. The whole world is indebted to the U. S. Coast and Geodetic Survey for its authoritative information about tides and currents.

The problem of determining the mean sea level, or mean low tide, is one that often confronts engineers, and they are not always fortunate enough, in many countries, to be able to obtain ready-made information from a reliable authority, such as the U. S. Coast and Geodetic Survey.

* See *Special Publication III*, U. S. Coast and Geodetic Survey, "Tides and Currents in New York Harbor," by H. A. Marmer, p. 77.

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‡ Received by the Secretary, October 27, 1927.

In preparing for an extensive final sounding, preliminary to an important dredging project in Isabela de Sagua Harbor on the northern coast of Cuba, the writer was confronted with the task of determining, in the shortest possible time, the plane of mean low tide. The datum, once selected, was to be accepted as final, for all sounding, for contract work, and for final measurements.

As no local information worth having was obtainable, mean low water during a month was found, and a correction introduced for that particular month. This was the zero or datum finally accepted for the work, and it proved to be (by a fortunate coincidence) the almost exact mean low tide for a series of years.

This matter of taking into account the monthly or seasonal variation of the level of the sea to obtain a mean of the whole year, received considerable attention. A record of tides and other meteorological information was started by the writer in 1911 and was continued under his supervision until the end of 1918. The record showed, since its beginning, the relative importance of monthly variation, on Cuban coasts, as compared with the mean range of tides. The mean low tide in January, 1912, was 0.33 m. (1.1 ft.) lower than the mean low tide in September of the same year. The mean range in Isabela de Sagua is 0.48 m. (1.6 ft.), so that the relative importance of the monthly variation is patent.

There is reason to believe that the same monthly variation occurs in Havana Harbor, where the mean range is only 0.28 m. (0.9 ft.). It is possible, therefore, that two independent observers, one in January and the other in September or October, may obtain results for the same tidal plane, which differ by 1 ft., or more; or that one observer may obtain, in a month, a mean high tide lower than the other observer has obtained in another month for a mean low tide. The year 1912 was extreme in that respect, the average of the eight years of observation giving 0.08 m. (0.26 ft.) below the mean of the year, in January, and 0.09 m. (0.30 ft.) above the mean of the year, in September and October.

The information obtained gave rise to a rule for determining the mean sea level, or mean low tide, which was to determine the average during a whole month and to apply to that value a correction, positive or negative, equal to the variation from the zero obtained in the same month at the Port of Havana, or the Port of Isabela de Sagua. If no observations were obtained from those standard ports (as is now unfortunately the case), a correction was to be applied, according to the month in which the observations were taken.

The Department of Public Works, now building the Central Highway of the Island, a modern road 700 miles long, has announced that it will establish bench marks, referred to mean sea level, on each kilometer post.

In the paper by Commander Rude, reference is made to this monthly or seasonal variation and the diagram (Fig. 19), giving the monthly means at Fort Hamilton, N. Y., from 1900 to 1909, inclusive, shows it plainly.* The author states: "Sea level as determined by a series of observations

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1100.

extending over only a month may differ as much as 1 ft. or more from that determined by another month". This covers very well the experience obtained at Isabela de Sagua.

The further statement* that: "For practical purposes a tidal plane obtained from a month of observations may be considered as fairly well determined", is not in contradiction with the other because it all depends on what is considered sufficient for practical purposes; but if taken separately, it may be misleading. Engineers basing their work on that authoritative source will continue to determine mean sea level or mean low tide from a month of observations and will believe they are "practical", leaving any further observations or corrections to the "theoretical" man.

The conclusion seems to be offered from the text that those monthly variations, although not always unimportant, cannot be determined on account of being due to differences of atmospheric pressure. The atmospheric pressure was determined with sufficient accuracy at Isabela de Sagua, and the diagram showing its mean monthly variation resembles, although in the opposite sense, that of the tidal planes. The highest pressure in January corresponds to the lowest level of the sea in that month and the lowest pressure in September and October corresponds to the highest level of the sea in those months. Further studies could not be made because exceptional months in the mean sea level could not be related to exceptional months in the atmospheric pressure, and thus no correction could be established for the mean sea level depending on the mean barometric pressure during the time of observations.

However, if the mean monthly variation of the sea level, depends on the difference of atmospheric pressure, this, in its turn, on account of its known regularity, must depend on some astronomical cause; and as such it is within the scope of the scientist. It is no doubt included in some of the elementary constituent tides, of yearly and semi-yearly periods, which are summed up in the wonderful tide-predicting machine of the U. S. Coast and Geodetic Survey.

Wind observations were made at the same time, in Isabela de Sagua, and were given considerable attention, both as to intensity and direction. Diagrams were thus obtained showing the average prevailing winds in each month.

This information about the wind is considered very interesting in harbor work. It may even show, in what month or season of the year, in a partly exposed location, it is advisable to proceed or to stop dredging or other harbor work. It also gives information about the best time of the year for soundings, with results that are sometimes contrary to popular belief.

Observations on atmospheric pressure, rain, and temperature were also made simultaneously.* These are subjects which also have their engineering aspects.

* The tabulated values and a general graphic of all the information referred to, have been published under the title, "Observaciones Mareográficas y Meteorologicas en el Puerto de Isabela de Sagua" ("Mareographical and Meteorological Observations at the Port of Isabela de Sagua"), in *Revista* de la Sociedad Cubana de Ingenieros, Sección de Obras Públicas, Vol. XI, (1919), September, October, November, and December. An article by the writer, with explanations and comments on the same, entitled "El Estudio de las Mareas y los Vientos en relación con las Obras de Puertos" ("The Study of Tides and Winds in Connection with Harbor Work") was published in *Revista* de la Sociedad Cubana de Ingenieros, Vol. XVII (1925), March–April.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

RE-ARRANGEMENT OF A BUSINESS DISTRICT: CHANGES IN RECENT YEARS IN PITTSBURGH, PENNSYLVANIA

Discussion*

By Francis J. Mulvihill, Assoc. M. Am. Soc. C. E.

Francis J. Mulvihill,† Assoc. M. Am. Soc. C. E. (by letter).‡—The paper by Mr. Schein is an important and direct contribution to the literature of city planning accomplishment. Because there are so few histories and progress reports of this art, the recorded experience at Pittsburgh, Pa., may suggest and stimulate additional response from "about five hundred cities and towns"§ in the United States engaged in city planning. The whole movement needs such accounts of accomplishment. The theory is fairly well established, but, in addition to knowing "how to do", "doing it" is important; so that practical demonstration lends impetus and acceleration to wise planning. Demonstration, too, will prove and serve to answer the question, "City planning is of what use?" The experience recorded by the author concerning the basic problem of transportation and circulation dependent on: (*a*) the street plan of the Pittsburgh Business District; (*b*) the engineering data of costs and materials; and (*c*) the overcoming of physical obstacles by demonstration of engineering skill, all suggest some considerations or comments. Those that follow are not critical, but are offered for the purpose of emphasis and expansion. The paper suggests relevant considerations: First, city planning procedure and accomplishment in Pittsburgh; and, second, the business district of Pittsburgh in the future.

City Planning Procedure and Accomplishment.—In general, the procedure followed and the accomplishment acquired are good. There are perhaps a

* Discussion on the paper by Nathan Schein, M. Am. Soc. C. E., continued from October, 1927, *Proceedings*.

† Formerly Planner and Asst. Chf. Engr., Dept. of City Planning, City of Pittsburgh; Landscape Archt.; Town and City Planner, Germantown, Pa.

‡ Received by the Secretary, October 10, 1927.

§ "Annual Survey of City and Regional Planning in the United States, 1926," by Theodore Kimball Hubbard, Annual Survey Number, *City Planning* (quarterly), April, 1927.

half dozen municipalities in the United States where substantial progress is being made, both in preliminary studies and actual accomplishment. Mistakes and inadequate planning cause great financial loss, entailing millions; but the annual budget of approximately $100 000 for the Department of City Planning of Pittsburgh is ungrudgingly provided by an appropriation ordinance of the City Council. No other city grants as much. In a sense it is just an insurance premium to prevent great loss. What is done now in public works is better done than it was a few years ago, and the budget amount is likely to increase. An historic background is a requisite for determining the rate of progress. Pittsburgh, like other cities, grew haphazardly and by chance from an unhappy aggregate of accidents in a series. Problems developed, consequently, and when they became serious, it was expedient to solve them. Prior to 1904 there was no comprehensive planning in Pittsburgh. Mr. Olmsted's report entitled "Pittsburgh, Main Thoroughfares and Down-Town District", in 1910, was among the first on city planning as it is understood to-day. It was only the year before, in Washington, D. C., that a meeting, the forerunner of the existing National Conference on City Planning, was held. The comprehensive Olmsted report stressed the importance of proper and adequate city planning. In 1911, the Pennsylvania Legislature passed the Enabling Act and the Department of City Planning of the City of Pittsburgh was created. The subsequent street improvements incorporated, almost in detail, the general Olmsted recommendations and the Department of City Planning established "follow-up". General studies were made by the City Planning Department for the relief of traffic congestion. In 1918, Diamond and Ferry Streets and Second Avenue were widened, and the Boulevard of the Allies was constructed. The Department of Public Works has much to do with projects of the character stated, because it is responsible for the engineering design and construction.

The supporting interest of the Citizens Committee on the City Plan is of so much value to the entire movement in Pittsburgh, that to treat its work and labors adequately would require the space devoted to the paper and more. Its report and study on playgrounds is indeed a classic of this element of city planning.

Since the paper was prepared, Grant Street has been widened from 60 ft. to 80 ft. With this widening completed, buildings of a height and architectural type new to Pittsburgh are being constructed and projected.

The Business District of Pittsburgh in the Future.—The distress of cities may be caused by difficulty in: First, making its life fit the mould of yesterday; and, second, trying to visualize to-morrow in the light of to-day's conditions. It is dangerous to attempt prophecy without the gift of vision. The Enabling Act of 1911, already mentioned, does not elaborate the functions of the Department of City Planning as does the recent Pennsylvania Act (1927). Until the present Act there was no authority at law that permitted the adoption of a thorough, studied, comprehensive, development, general design, or master city plan for the present and future guidance of Pittsburgh. The De-

partment of City Planning is, therefore, at work on the master thoroughfare and general design plans.

The author has mentioned changes that were accomplished a few years ago, and possibly there will be changes in the future. About the beginning of 1928 the Allegheny County projects of the Liberty Tunnels and Liberty Bridge will be open to traffic. These are a part of the Inter-District Traffic Circuit. The Department of City Planning has completed the preliminary design study of another link in the Circuit, the Crosstown Thoroughfare. This study has just been approved by the City Planning Commission. The widening of Grant Street has permitted a re-routing of trolley cars in a loop system. Every day, judging from congestion, the down-town streets look like Saturday afternoon, and the time is coming when they will look like the week before Christmas. Two-way streets will become one-way streets. Merchants are overcoming the fear of loss and the notion that all trolleys must pass their doors. Unlimited parking of automobiles has been replaced with limited parking or with none at all.

Streets cannot continue to be widened because of the cost. The people are realizing that the area in down-town streets is ample, but the best use of them is not practiced at present. The cost of delays to the owners of trucks, buses, taxicabs and automobiles, conservatively estimated, now is $25 000 daily. These owners will avoid the delays and effect the saving. A railroad will abandon or salvage its equipment along Duquesne Way and in "the Point"; sell its land in these locations; acquire the site and provide better modern facilities elsewhere, without any financial loss, from the proceeds of the old plant. The Point, now backward, will have a new life and business will expand into that area. The flood will be controlled and the rivers' fronts improved. The down-town traffic circuit will be operating, gyratory, one-way, along Duquesne Way, Water, Grant, and Eleventh Streets. Within the circuit there will be no vehicles, unless in an emergency, during the day. Trolleys will be in a subway loop and off the street surface. Buses and cruising taxicabs will have stations and terminals off the streets. Streets for deliveries, collection, and merchandising services will then be used at night. Vehicles will be absent from the streets down town, not because they cannot get there or are prohibited, but because of the economic loss to the owners if they attempt it. The streets will again be free to the people when every day is Christmas in Pittsburgh.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE LAKE WASHINGTON SHIP CANAL, WASHINGTON

Discussion*

By Joseph M. Clapp, M. Am. Soc. C. E.

Joseph M. Clapp,† M. Am. Soc. C. E. (by letter).‡—The writer had the Lake Washington Ship Canal as his home work, under the direction of the District Engineers, for the ten years from 1901 to 1911; gathered much of the physical data used in solving the problems in relation to the project; is familiar with the locality and its geological characteristics; and takes pleasure in adding a few details relating to this great project.

Something has been said of the late Colonel Cavanaugh's connection with this project and the manner in which he directed it. It is only fair that greater stress should be laid to the fact that much pertaining to the details of design and the actual work of construction was done by Mr. Sargent, the very efficient Engineer Assistant. It was his untiring efforts that made possible the economical design of the system and of the plant and its operation during construction; all of which resulted in the very low cost of the locks and dam as a whole.

It has been stated§ that the lowering of Lake Washington and the outflow of the Cedar River, *via* Ballard and the locks, has lessened the flood problems in the Duwamish Valley. While this is so, it is not as great a factor as the divorcement of the waters of the White River from the Valley of the Duwamish to its new channel *via* the Puyallup Valley to the Sound. The run-off from the White River shed during flood seasons is several times greater than that from Lake Washington and its tributaries.

A hard clay formation is the bed-rock of the country and, once it is well defined, is considered sufficient to support the structures that are built in this section. To use piles under the walls where this clay formation is met, if

* Discussion on the paper by W. J. Barden and A. W. Sargent, Members, Am. Soc. C. E., continued from October, 1927, *Proceedings*.

† Cons. and Contr. Engr., Seattle, Wash.

‡ Received by the Secretary, September 27, 1927.

§ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1231.

the weight is less than 5 tons per sq. ft., is superfluous and may result in poor construction for the reason that, to drive wooden piles to any penetration into the clay means to upset the fiber, cause the pile to telescope, and thus destroy its bearing value for heavier weights. Steel or reinforced concrete piles would have to be used if greater weights were to be supported.

In the early days of this work it was the writer's duty to direct the excavation of a ditch so as to bring the outlet of Lake Union through a channel located wholly on the canal right of way and under the control of the War Department. The Northern Pacific Railway crossed the right of way at a point north and west of Fremont, Wash. The Railway Company constructed its trestle crossing in the dry and before excavation in its vicinity was begun. On working the excavation under the trestle many of the piles were exposed, and it was found that they had telescoped and were of no real value. The crossing had to be rebuilt, and sills and posts were substituted for the piles.

At the portage separating Lake Union from Lake Washington, where the narrow channel was provided for the passage of logs in 1888, it might be mentioned that at first a lock was actually provided, 50 ft. long, 18 ft. wide, and 4 ft. deep, for the passage of small steamers. Several lockages of steamers were made, but there was no profitable business. Eventually, the locks were allowed to decay and a raceway for the passage of logs was constructed and operated on a toll basis by the Lake Union Mill Company.

In determining the location for the canal, the routes through Salmon Bay were canvassed and the present route determined by a Board of Engineers, all prior to 1896, when the writer first became familiar with the project and the District Office was established at Seattle. Later, about 1906, another route was canvassed to enter Lake Washington by way of the Duwamish Valley, with a canal to be dug through Beacon Hill and the territory intervening between the Duwamish Valley and Lake Washington, about on line with Spokane Avenue.

The plan contemplated the excavation of a canal between Lake Washington and Lake Union and either the raising of the waters of Lake Union to the level of Lake Washington; lowering Lake Washington to the level of Lake Union; or lowering one and raising the other to some intermediate point. This plan was advocated by a company interested in the reclamation of the tide-flats at the mouth of the Duwamish River, known as the Lake Washington Canal Company.

The cross-country canal involved excavation through a hill about 130 ft. high and the moving of a great many million cubic yards of material. It was the writer's duty to make comparative estimates for the information of the Board of Engineers that judged this work. After careful consideration the Board discarded this route, for obvious reasons, in favor of the present very logical route, which had been previously determined upon by other Boards of Engineers.

Some consideration was also given to a possible canal by way of the Black River and the Duwamish River to Puget Sound. Thus, it will be noted that careful consideration was given the matter of routing this canal.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

BASIC INFORMATION NEEDED FOR A REGIONAL PLAN

Discussion*

By Messrs. Charles W. Eliot, 2d, U. N. Arthur, and Charles Wellford Leavitt.

Charles W. Eliot, 2d,† Esq.—In Washington, D. C., the National Capital Park and Planning Commission is generally following the program laid down by Mr. Lewis in his most interesting paper; but there are two exceptions.

In the first place, he suggests that the first step in regional planning work should be the definition of a boundary.‡ The Commission has carefully avoided setting any limits, and is going to avoid it as long as possible because it feels that the extent of the influence of the Capital City is a very indeterminate factor and something that will bear a great deal of study before any such limit is set.

In the second place, it has departed from the suggestions in regard to "open spaces". In Washington, a different set of indications are used from those suggested by Mr. Lewis. The idea has been to indicate areas withdrawn from urban occupation. That includes both private and public areas in very much the same way that Mr. Walker has outlined,§ but it is, perhaps, a little more accurate definition of "open spaces" than just those two words.

The City of Washington was very fortunate in the equipment that was available at the start for planning work. The U. S. Geological Survey, the U. S. Coast and Geodetic Survey, and the Air Service have made a great many studies and many maps of the Washington region. The basic maps, drawn to a scale of ½ mile to the inch are found to be entirely satisfactory. For more detailed studies, there are remarkably accurate maps of the District of Columbia at 400-ft. scale and general street maps at 1 600-ft. and 1 000-ft. scales.

* Discussion on the paper by Harold M. Lewis, M. Am. Soc. C. E., continued from November, 1927, *Proceedings*.

† City Planner, National Capital Park and Planning Comm., Washington, D. C.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1506.

§ *Loc. cit.*, November, 1927, Papers and Discussions, p. 2380.

In most regional planning work the studies are entirely separate from the city planning studies; but, in Washington, that is impossible, because the National Capital Park and Planning Commission has particular executive functions in the District of Columbia. It has the power and money to purchase land for parks and it has the guardianship of the "Highway Plan", which controls the platting of sub-divisions. Congress has seen fit to give it power to make regional studies, but it is forced constantly to meet immediate problems of design and planning within the District of Columbia. Although the attempt is made to keep the regional planning studies in a fluid state while data are being collected, the Commission is constantly having to make decisions in the District of Columbia that affect the regional plan. That is another reason why it is trying to keep the boundaries of its region somewhat indefinite because there is no knowing how far immediate decisions are going to influence the regional plan. Admitted that this method of going at the work is unfortunate—to have to be controlled by immediate decisions—that is the state of affairs.

Regional planners in Washington are grateful for the work that has been done in New York and particularly for Mr. Lewis' helpful suggestions; and hope, if they can get the co-operation of engineers, landscape architects, and all others interested in city planning, to be able to produce a plan, for the National Capital and the region around Washington, that will be a serviceable guide for the growth of that community and an inspiration for the remainder of the country.

U. N. ARTHUR,* ESQ.—Among the various outstanding problems confronting the great centers of population probably none is of greater moment than the development of a comprehensive plan. Regional planning, being still in its infancy, will measure its accomplishments by the basic principles governing the method of attack. Therefore, it is fortunate, indeed, to have the subject outlined as clearly and concisely as in this paper.

A finer opportunity is afforded for the planning of the large area of a region or district than that of city areas. This follows from the fact that, before any attempt had been made at scientific city planning, the area within the incorporated city limits was often largely platted and sufficiently developed to make impracticable any radical change in the system of major thoroughfares, parks, playgrounds, etc., while, for regional work, large areas are often open and susceptible of being properly planned to care for the various factors essential to the growth of the community.

While there can be no question as to the wisdom of supplying the various types of data suggested by Mr. Lewis,† it might be well to emphasize the necessity of securing the basic physical, or land, facts of the area before any concentrated effort is made to produce a regional plan.

The three fundamental factors that a community of any size has to deal with are: (*a*) The land it occupies; (*b*) the people that dwell in the land; and, (*c*) the social and industrial development. The other factors influencing the

* Chf. Engr., Pittsburgh Planning Comm., Pittsburgh, Pa.

† *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1507.

study and development of the plan are: First, general information; second, provision for making changes easily; and, third, means for accomplishing the final object—the plan.

The willingness with which planners often accept general information as sufficient for planning purposes has been a principal source of criticism. Often the weak points in a finally submitted plan are exposed to the attack of critics who claim that all planners are dreamers and that money expended on the study and development of a city or regional plan is money wasted. The exigencies of the case often require that the broader the extent and scope of the planning operation, the more generalized is the information secured. Such information, spread too thin, means vague information, which is frequently deceptive and leads to the development of plans that are impractical and expensive and often cast doubt on the honest intent and purpose of the planner.

It is a truism that, in most cases, the sites of cities and present centers of population were not selected because the physical surroundings were adaptable to easy development, but because of the existence of important crossroads; or because of the transportation facilities afforded by navigable rivers; or, because of natural harbor and port facilities. Such situations often have immediate surrounding topography of the most rugged nature. It seems, therefore, that, in the first analysis, basic information is the securing of physical facts relating to the land—geodetic, geologic, and topographic. It is true that at the initial conception of most planning undertakings, ample funds have not been available for securing the engineering data so essential to the proper and economic completion of the work. However, if those data were properly evaluated and their importance recognized, in most cases, provision would be made for assembling them as a part of the planning program and as the logical first step toward real planning progress. The first fundamental consideration that any community has to deal with is the land it occupies. It is not only the first, but it is perpetual. Bridges and viaducts may be built across rivers and valleys; hills may be tunneled; extensive grading operations may be carried out; but regardless of the magnitude of these engineering feats, the terrain is altered only to an insignificant degree. Whatever be the location of the site, that it remains. The social, economic, and industrial development of the location and the chances for the existence and happiness of the population depend exclusively on the logical and scientific development of the site.

The basic physical, or land, facts are, and always will be, unavoidable. If they are ignored during the planning stage of any undertaking, they will be encountered during the construction stage. Disregarding these facts results in experimental building, and the structure (street, bridge, viaduct, tunnel, public building, sewer, etc.) that does not satisfactorily meet the physical conditions is inevitably replaced by one that does. Frequently, the cost of relieving one situation that has become insupportable is more than sufficient to have paid for securing basic data to serve the whole region.

As a specific example, illustrating both the necessity for, and economy to be gained by, a regional plan, the speaker believes no better case can be cited than is afforded by Allegheny County, Pennsylvania. This county has an area of 732 sq. miles. Within its boundaries are 4 cities, 70 boroughs, and 51 townships, many of the latter having legislative functions similar to those of the boroughs. The City of Pittsburgh is approximately at the geographical center, and being the largest and oldest of the municipal corporations, dominates the regions, many of the boroughs being simply dormitories for the city. (See Fig. 4.) The need for a regional plan has been felt for many years and various suggestions for accomplishing the co-operative solution of the municipal problems affecting this area have been proposed.

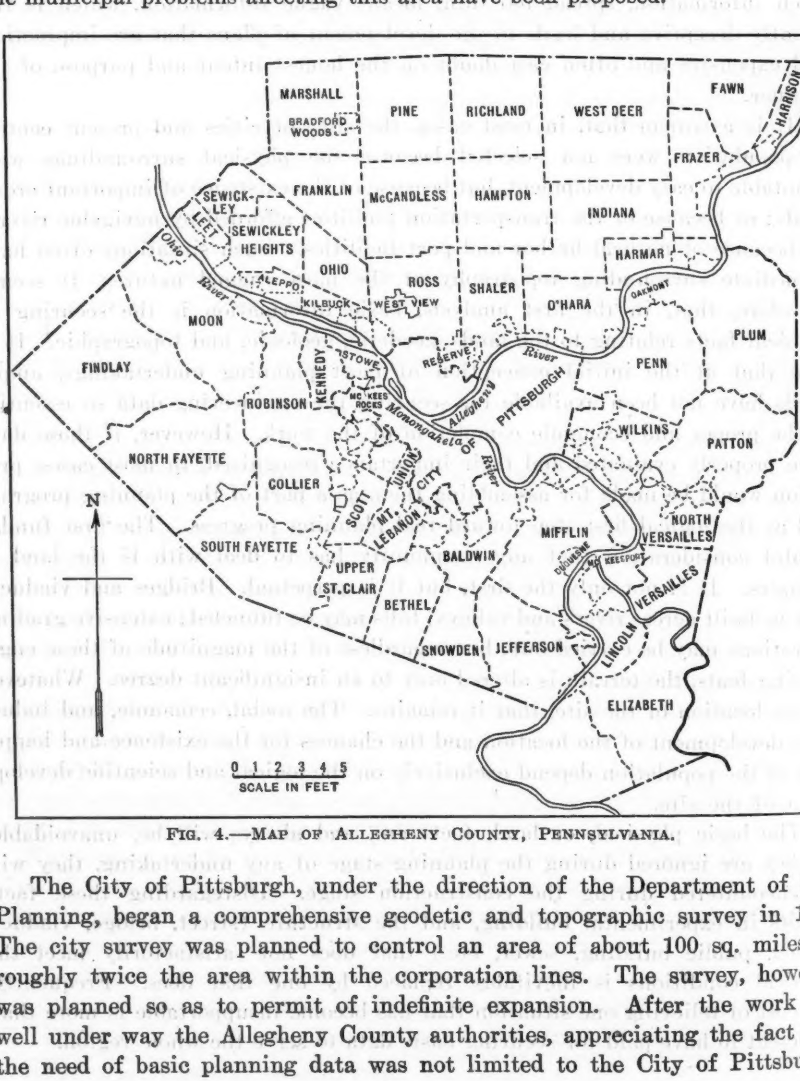

FIG. 4.—MAP OF ALLEGHENY COUNTY, PENNSYLVANIA.

The City of Pittsburgh, under the direction of the Department of City Planning, began a comprehensive geodetic and topographic survey in 1923. The city survey was planned to control an area of about 100 sq. miles, or roughly twice the area within the corporation lines. The survey, however, was planned so as to permit of indefinite expansion. After the work was well under way the Allegheny County authorities, appreciating the fact that the need of basic planning data was not limited to the City of Pittsburgh,

but was equally essential in carrying out the extensive construction program of the County, entered into a contract with the City to extend the survey to control the entire county.

This survey is of a somewhat unusual character and embraces the essential information needed for a regional plan. The general schedule adopted provides for the following:

1.—Precise triangulation extended over the metropolitan area. The stations of this triangulation average about 1 per square mile in incorporated or built-up territories, and about 1 for every 2 sq. miles throughout the remainder of the county. The accuracy of the triangulation is such that the probable error of any distance will not exceed 1 in 100 000.

2.—Precise traverse amplifying and making usable the results of the triangulation. The stations of the traverse are being well "monumented" and referenced. They coincide, as far as practicable, with existing property monuments. The traverse will have an accuracy represented by a limiting closing error of 1 in 20 000.

3.—Precise levels with bench-marks established on permanent structures along the routes and on traverse and triangulation monuments.

4.—Topographic sheets of the city area on a scale of 1 in. equals 200 ft.

5.—A property map of the city area on a scale of 1 in. equals 50 ft. This is to be the base map for all departmental information where dimensions are required.

6.—A wall or desk map on a scale of 1 in. equals 1 000 ft., compiled from the data shown on the large-scale topographic maps.

Topographic Survey.—Description.—The topographic map, which is being made of the Pittsburgh Area, is a definite and valuable result of the survey. On it are shown to exact scale and position all streets, roadways, curbs, retaining walls, steam and electric railroads and bridges; public, semi-public, and industrially important buildings; as well as streams, lakes, etc. The conformation and elevation of the surface of the ground are shown by contour lines. From the topographic map grades may be accurately computed, drainage areas may be scaled within a fraction of an acre, and excavation quantities computed. The Pittsburgh map sheets are published on a scale of 1 in. equals 200 ft., with a 2.5 and a 5-ft. contour interval. The datum plane for contour elevations is mean sea level, thus placing the maps on the ultimate datum plane and agreeing with Federal, State, and other maps.

The publication scale of 1 in. equals 200 ft. was adopted as best fitting the conditions imposed, which are:

1.—The map scale must be large enough to indicate clearly and accurately all the information desired.

2.—The scale should bear a definite relation to the accuracy of field measurements, and, since the majority of the positions are located by stadia, and stadia distances are dependable only within 1 ft., the scale should be such that distances may be plotted and scaled to the nearest foot.

3.—The scale of the map must be small enough so that the area covered by any one map sheet would be of such size that ordinary district improve-

ment projects may be covered and studied upon it, without the necessity of assembling and matching the various adjoining sheets.

4.—From an economic standpoint it is desirable to keep the scale as small as possible, in order to decrease the cost of the field work and of reproduction.

While the scale of publication is 200 ft., the field scale, on which the maps are sketched in the field, is 165 ft. The purpose of this is the possible increase in accuracy of scale by the reduction, which is a part of the photographic process of publication, and the sharpening of lettering and other cartographic details and consequent improvement in appearance of the final published copies. It is of further advantage to make the field scale as large as is consistent with stadia methods for the purpose of making enlargements. For instance, it is possible to make a better photographic enlargement to a scale of 100 ft. by using an original copy on a scale of 165 ft., than would be the case if the original were on a 200-ft. scale.

Map Projection.—In deciding on the system of projection to be used on the map sheets, there were three conditions to be considered:

1.—The projection must be such that the maps will always be in correct orientation; that is, the boundaries of each sheet should always be true north and south, east and west, in order that the true azimuth or bearings of any line may be quickly scaled from the map.

2.—The projection should permit of practically indefinite expansion.

3.—In order to be joined correctly and easily to other maps, such as those published by the Federal Government, State, Interstate Commerce Commission, etc., the boundaries of each map sheet should be some even value of latitude and longitude, these values being co-ordinates based on the North American datum.

4.—For ease and simplicity of ordinary distance, bearing, and position computations, the maps should have superimposed upon them a rectangular system of co-ordinates.

Reproduction.—After completion, the field sheet is reproduced in four colors. Cadastral information is shown in black, contours in brown, drainage in blue, and public property in green. The reproductions are to exact scale within the limits of paper expansion, and registration is correct at all points, the allowable registration error being 0.01 in. Two hundred copies on light-weight paper of these four color reproductions are printed on the initial order. In addition, ten copies are printed on "lenora" cloth for use in binding in atlas form. It is also expected that several copies will be secured showing the cadastral information only.

Estimates.—The estimated unit costs for the completed survey are, as follows:

Triangulation:

City area.........$350 per sq. mile, including monuments and computations.

County area.......$300 per sq. mile, including monuments and computations.

Precise Levels:

City area.........\$70 per lin. mile, including the setting of bench-marks and computations.

County area.......\$60 per lin. mile, including the setting of bench-marks and computations.

Precise Traverse:

City area.........\$100 per lin. mile, including monuments and computations.

County area.......\$90 per lin. mile, including monuments and computations.

Topography:

City area.........\$1 800 per sq. mile, including reproduction.

County area.......\$1 500 per sq. mile, including reproduction.

CHARLES WELLFORD LEAVITT,* M. AM. SOC. C. E. (by letter).†—Any one not familiar with the difficulties that have attended the gigantic undertaking of "planning New York and its environs" will find it almost impossible to compare this project with anything in regional planning taken up elsewhere, as there is no gauge that can be used to compare New York with any other city. Mr. Lewis has endeavored to stress this point with which he is perfectly familiar, but it cannot be emphasized too much for the reader. The paper very properly endeavors to give a clear idea of what basic information is to be secured.

The question arises as to what difference there is between a regional plan and the ordinary development of a city's growth under a mayor, common council, boards of water, sewer, health, streets, parks, buildings, etc., directed by a city engineer, with his various departmental assistants who, until very recently, have directed the growth of the cities of the world.

It is possible to infer from Mr. Lewis' paper that, the "basic information" having been secured, there is contemplated "research work" upon the various items under which this basic information has been classified and which in the past might properly have been studied by the city engineer. The fact is that while some information has been gathered by various city departments, as to sizes of water and sewer pipes, sanitary measures, street widths, park areas, building heights and strength, and fire-proofing; yet information is lacking as to whether it will be better to carry water in large mains to different satellite communities and there distribute, or to add to them from time to time, and some day be confronted with the necessity of scrapping the whole system and starting off afresh, which is an expensive method. In these old records, there was not a sufficient knowledge of future drainage and sewerage necessities to make it possible to plan the whole region with confidence, and to build what was immediately required, with assurance that, in the future, the work done would all come together and make a permanent system that would take care of any growth.

Each community acted independently; each section of the metropolitan center solved its individual problems; with the result that most all the work done will prove inadequate for the future and a whole new system must be

* Civ. and Landscape Engr. (Charles Wellford Leavitt & Son), New York, N. Y.

† Received by the Secretary, October 25, 1927.

built at great cost; and so on through the parks and building departments, the streets and highways, the docks, transit lines, areas set aside for manufacturing industries, business, and residences, etc.

In Europe, and in America's older cities, these troubles were not so apparent, because the growths were slower and frequently pipes were worn out before they had to be enlarged; but park areas did not grow of themselves nor did streets become wider; and as early as 1910 the people became aware of the facts, and their interest became fixed upon research to determine how, in the future, they might avoid waste and provide for the comfort of city dwellers.

City engineers are usually overworked with many details of daily importance and have not the time and are not given the facilities to delve into the future or to determine what may take place in 50 to 100 years. Here, then, is where the city and regional plan will show an economic return to a county region, or a city, by securing the necessary power to plan ahead and make it possible to use the land for the purpose to which it is best suited.

This will take time. The first step is to collect the basic information, which can only properly be done by some one with experience and knowledge of what is needed. The second step is to interest the various official bodies contained within the region to be planned; third, to bring these official bodies to a point where they will work together for the common good; and fourth, to form an organization that will endure indefinitely either to carry out the plans, or to change them to meet any new conditions that may develop in the region's growth.

With such a program definitely outlined, and with a director versed by experience in what is and what is not needed, this basic information may be gathered, classified, and used to advantage. Most undoubtedly it will be well worth while for any community with a rapid growth, or which may anticipate such a growth in the future to have available this basic information. On the other hand, the amassing of information that cannot be classified, or that cannot be made available for use either because of its volume, or the manner in which it is collected, may prove a veritable menace. Often it tends to confuse and discourage the ruling bodies, which may properly, if they do not understand the plans and program, roll them up and put them out of the way.

The great danger in collecting basic information for a regional plan is in getting too much and not being able to digest it into a clear workable plan that will carry its own conviction.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

FORECAST: THE REGIONAL COMMUNITY OF THE FUTURE

Discussion*

By Charles Wellford Leavitt, M. Am. Soc. C. E.

Charles Wellford Leavitt,† M. Am. Soc. C. E. (by letter).‡—Throughout this paper one senses the fact that the author has built up his forecast largely upon the work done on the "Plan of New York and Its Environs". Therefore, in reading it, one should realize that there is little, if any, comparison between New York and any other city; that the forces acting in the creation of New York will, in all probability, act differently upon other metropolitan areas; and that the regional communities of the future will be unlike the regional community of New York.

"Regulation of city growth" is not always possible. The forces making for city growth are human, and while they are amenable to certain laws and regulations by the police for the general health and welfare of the community, they are not always willing to follow, for instance, the directions laid down by a regional plan. The success which the city builders are after may depend on their originality, backed by their energy; and those drawing up a regional plan should make it sufficiently flexible to take care of these city builders and not block them in their ventures, except when their acts might endanger the lives, health, and general welfare of the country.

For instance, if a plan of New York had been prepared some years ago it would undoubtedly have provided for large manufacturing plants, whereas, in the past twenty years, the author states that plants have been growing smaller although more numerous.

Guiding or regulating the location of people's homes into the so-called blighted districts where there is now perhaps only 1% density of population,

* Discussion on the paper by Thomas Adams, Esq., continued from November, 1927, *Proceedings*.

† Civ. and Landscape Engr. (Charles Wellford Leavitt & Son), New York, N. Y.

‡ Received by the Secretary, October 28, 1927.

in order to avoid the congestion of certain other popular sections, may be a very laudable, though difficult task. Forces other than general direction or regulation must be called upon, and here is one of the great problems of the city planner, to study the psychology of the builders and provide what may be necessary to attract them into the blighted districts.

The quotation from Hilaire Belloc* is pertinent, and should be taken to heart. It should be possible, with all the enlightenment of past ages and present rapid growths, to develop plans that would stabilize city realty values and avoid what has recently become so prevalent, namely, the moving of commerce, manufacturing, business, shopping, and residential districts so rapidly from one section of the city to another that good buildings have not time to wear out, but must be sacrificed to the movement of one district into the fashions calling for another; and quite frequently the abandonment of the building suitable for one purpose, to the wretchedness of an area which has been blighted by this movement.

The cities of the old world are far more stable in character than American towns, built and occupied by a cosmopolitan race whose great objective is to speed up and arrive somewhere "where they aint", with a pot of gold in hand.

Such motives do not encourage stable real estate investments, which must come about from a population more satisfied and slower to move; although on the other hand, without movement, there results stagnation which may be even worse than constant movement. The ideal would be one of reasonable activity with a general tendency toward better conditions in both residences and business.

Water-front development is again peculiar to seaboard, lake, gulf, and coast towns and does not apply to some of the inland metropolitan regions, which also have a future.

Can any other section of the world compare with the Park Avenue District in New York? However, "we do find in such cities as Atlanta and Kansas City problems of growth just as puzzling"; but what has helped in studies of New York does not aid in these cities but rather tends to lead astray.

Again, in the quotation from Mr. Richard M. Hurd in regard to the returns on low *versus* high building,† is found an argument most upsetting to smaller towns where a big price will be paid for the sky-scraper; and it is nearly impossible to convince any one that a sky-scraper is harmful to a town; for, while the writer feels with Mr. Adams and Mr. Hurd, that the building of tall structures is apt to hurt, yet in many instances towns have remained asleep until the tall building came to wake them into activity; but it did not become epidemic as in New York, Chicago, and Detroit. A hard and fast rule may not work for the smaller metropolitan regions.

Anything that may tend to re-establish a demand for land in these blighted areas is bound to be of great assistance, not only in that immediate neighborhood, but throughout the entire region. These blighted districts are usually equipped with sewers, water, electricity, and, frequently, paved streets, in which the city has a large investment without any chance to recapture it

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1537.

† *Loc. cit.*, p. 1538.

except by activity and building. Otherwise, this city money lies without return in adequate taxes, just as a sub-divider, who has equipped his land with all street improvements, is eaten up with interest if he does not sell his lots promptly.

Leaving vehicular traffic on the surface and putting wheeled transportation beneath and pedestrians above, may help a little for a time, but will not the section, so treated, soon again be overcrowded with no form for relief and thus in the end "confusion worse confounded" reign? Has all this not proved that the best way to produce reasonable living conditions, as well as areas in which to work, is by decentralization with careful thought given to building code and zoning?

Into the rural communities, along the main arteries of travel, is now pushed a ribbon-like development, making the roads between large cities such as New York, Boston, and Philadelphia, almost a continuous row of citified houses, and one wonders if the predictions of the author of "Road Town", published about 20 years ago, is not to be realized. In England, this ribbon development is giving the lovers of rural attractions considerable concern, because such rural drives as were enjoyed in the past, have ceased to exist in many cases and one drives from city to city through an apparent city.

Will the regional plan of the future correct these blighted districts, congested areas, and sky-scrapers, and relieve the owners of single-family houses from looking directly into the window of their neighbors? Will some of the country outside the city remain rural? These are points over which the intelligent citizen must pause, and if such regional plans as have been drawn give promise of relief, then the citizens of seaboard, lake, gulf, and coast towns as well as cities inland, will demand that they too be included in the comprehensive plan for the future.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

HISTORIC REVIEW OF THE DEVELOPMENT OF SANITARY ENGINEERING IN THE UNITED STATES DURING THE PAST ONE HUNDRED AND FIFTY YEARS

A SYMPOSIUM

Discussion*

By Messrs. James W. Armstrong, J. W. Ellms, Caleb Mills Saville, W. Kiersted, M. M. O'Shaughnessy, John H. Gregory, Karl Imhoff, Kenneth Allen, L. L. Tribus, Arthur E. Morgan, Arthur M. Shaw, W. B. Gregory, H. F. Gray, Thornton Lewis, and Samuel R. Lewis.

James W. Armstrong,† M. Am. Soc. C. E.—Touching as it does almost every phase of the water-works problem this is an interesting paper. Mr. Fuller has shown very clearly the value, to humanity, of a pure water.

One or two points have recently come under the speaker's observation that might be of some interest in connection with pipe coating. He recently had occasion to inspect the inside of a 7-ft. steel water main that had been in service about eleven years. It is laid under 25th Street in Baltimore, Md., and is protected on the outside by a concrete casing and on the inside by a coating of bitumastic enamel.

A walk of about ½ mile and return through this line showed it to be entirely free from tuberculation. The only defects discovered, were around the rivet heads in the bottom of the pipe. The evidence is that workmen, in passing through the pipe line, had knocked the protective coating from some of the rivets and corrosion had set in at such places.

* This discussion (of the Symposium on Historic Review of the Development of Sanitary Engineering in the United States During the Past One Hundred and Fifty Years, presented at the meeting of the Sanitary Engineering Division at Philadelphia, Pa., on October 6, 1926, and published in September, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Filtration Engr., Baltimore City Water Dept., Baltimore, Md.

If bituminous enamels are properly put on they make an excellent waterproof coating for both steel and concrete, but as they have to be put on hot, it is necessary to have the surface to which they are applied clean and perfectly dry. Failure of bitumastic coatings can generally be traced to neglect on the part of workmen to comply with these precautions.

A piece of casting taken from a broken water main in New Orleans, La., revealed the fact that the interior was nearly free from corrosion and that a thin smooth coating of lime had been deposited, which formed a natural and effective protection. This would indicate that cities using lime for softening purposes have little to fear from tuberculation of cast-iron pipe.

In Baltimore, where the water is naturally corrosive, sufficient lime is added to the filtered water to maintain a pH value of about 8.2 for all water entering the city mains. The alkalinity is thus maintained at a point sufficiently high to reduce its corrosive qualities very greatly. Cities annoyed with the corrosion of pipes might avoid much of their trouble by raising the alkalinity of the water delivered to the city mains.

In tracing the history of the water-works pump, Mr. Fuller has shown that the electrically operated centrifugal pump has, after a long struggle, come into its own. There are three great advantages in pumps of this kind: (1) low first cost; (2) very little repair work; and (3) an absolute minimum of labor for operation. At Montebello, the low-lift pumping station contains five electrically driven centrifugal pumps, having an aggregate capacity of about 250 000 000 gal. per day. For more than eleven years, practically all Baltimore's water supply has passed through this pumping station, and there is no other power available, should the electricity fail. For several years, after first placing the station in service, power was supplied over a single underground cable. On one occasion a blow-out in this cable caused considerable uneasiness, but it was repaired before the storage in the 15 000 000-gal. reservoir ran out. A second cable was afterward installed, and since then there have been only momentary interruptions in the power supply. During the entire time this station has been in operation, only one man has been employed on each shift of eight hours, except for repair work.

The cost of pumping water, neglecting fixed charges and overhead, from 1916 to 1919, against an average head of about 40 ft., was $1.62 per 1 000 000 gal. During the entire life of the plant, more than eleven years, the cost averaged about $2.10 per 1 000 000 gal. for pumping against heads ranging from 30 to 43 ft. The high price of coal during the World War was the principal cause of the increased total cost.

The plant at Little Falls, N. J., should be considered the prototype of all modern filter plants, as it was the first to be entirely constructed of reinforced concrete. The use of reinforced concrete for such purposes spread very rapidly. The great variety of forms and shapes into which it could be moulded, led many engineers to believe that its use was the solution of most of their structural problems and has made them a bit slow about recognizing its limitations. Lately, many have learned that concrete, passing as excellent when fully protected from the elements, may fail utterly when exposed to the weather for a few years.

It has also been learned that concrete through which water is leaching is doomed; that the passage of water through concrete will, in turn, dissolve both the calcium and aluminum compounds; and that the dissolving of these compounds robs the concrete of its strength and leaves it an easy prey to the action of frost and the elements. To secure imperviousness for all water-bearing or exposed concrete is of the utmost importance.

If field operations could be carried on with the precision of laboratory work, it might be possible to secure satisfactory concrete, but the difficulty of controlling the human factor may make futile the most painstaking and careful work in securing a proper quality, grading, and mixing of materials. Unless some method of securing denser concrete can be obtained, engineers of the future may come to rely more on some outer coating to render concrete impervious.

In the ordinary distribution system valves are not operated often, and an old Superintendent might, after years of service, not have had experience enough to judge rightly the merits of a particular valve. With the advent of large mechanical filter plants where valves are opened and closed every day, certain weaknesses have developed that had not been realized. Special efforts should be made to improve the details of valves designed to operate in a horizontal position.

J. W. Ellms,* M. Am. Soc. C. E. (by letter).†—The survey by Mr. Fuller of the progress made in the past half century in the water-works field, is both interesting and instructive. To the older members of the Society, whose professional activities have run parallel with the development of the art, this record may seem commonplace. However, does it not naturally raise the question of how such improvements have been made possible, and to what specific cause the advance may be attributed.

The general increase in scientific knowledge is doubtless a partial answer to the question; but is it not more specifically answered by the vast amount of research work which has been carried on in this field during the past fifty years by professional men? The labor of individuals and of associated groups for the solution of specific problems has furnished the real basis for these improvements. The large sums of money expended in research work have not always been appreciated; but without these funds the present status of the art could not possibly have been reached. State, municipal, and private funds have enabled scientists to cope with the problems arising in this field of applied science. The benefits that have been derived from systematic research in the past is a sufficient guaranty of what may be accomplished in the future. Co-operative research during the World War certainly proved what can be done under the stress of necessity.

While the writer realizes that much is now being done by National engineering societies in furthering research work, he believes that more may be accomplished and greater advances in the art secured by more active and properly directed group action. The field is broad and the rewards ample for

* Engr., Water Purification and Sewage Disposal Dept., Public Utilities; Cons. San. Engr., Cleveland, Ohio.

† Received by the Secretary, September 19, 1927.

all those who have a genuine interest in the progress of applied science in water-works engineering. The problems to be solved involve the fundamental branches of science, such as physics, chemistry, and biology; and in applied science, the arts of civil, mechanical, and electrical engineering. Where among human activities could one find a more fascinating or varied field for labor? To the younger members of the profession, it should offer a life work of usefulness.

Caleb Mills Saville,* M. Am. Soc. C. E. (by letter).†—Mr. Fuller has performed notable service to engineers and others specializing in water supply work in bringing together in such a concise but comprehensive manner this critique of water-works practice during the past century and a half.

While probably the great fires in Chicago, Ill., and Boston, Mass., in the early Seventies furnished a great impulse to water-works construction, it is notable that many of the works in Northern cities were started about ten to fifteen years earlier. For some reason, the late Fifties and early Sixties seems to have been a time of considerable water supply work in this section, possibly because of growth in population and increase in value of property.

Apparently, the largest increase in public water supply works came after 1880. Of the 9 850 plants estimated as in service in 1924 and given in Mr. Fuller's Table 1,‡ 6.1% were installed prior to 1880; 32.5% prior to 1896; and 67.5% since that date. That is, about two-thirds of the total number of water supply systems in the United States, as estimated in the "Manual of American Water Works", were installed between 1896 and 1924.

The use of house cisterns or tanks connected with the public supply was a very common mode of supply for individual buildings, because they could be filled at night when the city draft decreased. This reserve helped during the day when the city supply, due to inadequacy of trunk lines, was unsatisfactory on the higher elevations. Previous to this, and in the transition period from dug wells to public supply, many private houses had water-tight cisterns in their cellars, in which rain water from the roof was stored and pumped by hand-operated force pumps to tanks in the upper part of the house. Often these cisterns were supplied with charcoal filters through which the water was passed before being used. In many towns and cities, about a century ago, large public cisterns were located in various parts of the municipality, in which water was stored for fire protection purposes. Such cisterns were existent in Malden, Mass., and Hartford, Conn., up to within a few years, although abandoned many years ago, probably with the introduction of aqueduct water. The locations of these cisterns were often forgotten and only recalled when they were accidentally uncovered or broken through by heavy loading.

Wooden pipes of bored logs were used in many localities. They were in service for some years in Boston and elsewhere. In Hartford, such pipes were used as early as 1803 in at least two supply systems bringing water to the center of the city from springs several miles away. On Fig. 1 is shown a

* Mgr. and Chf. Engr., Hartford Water-Works, Hartford, Conn.

† Received by the Secretary, September 27, 1927.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1588.

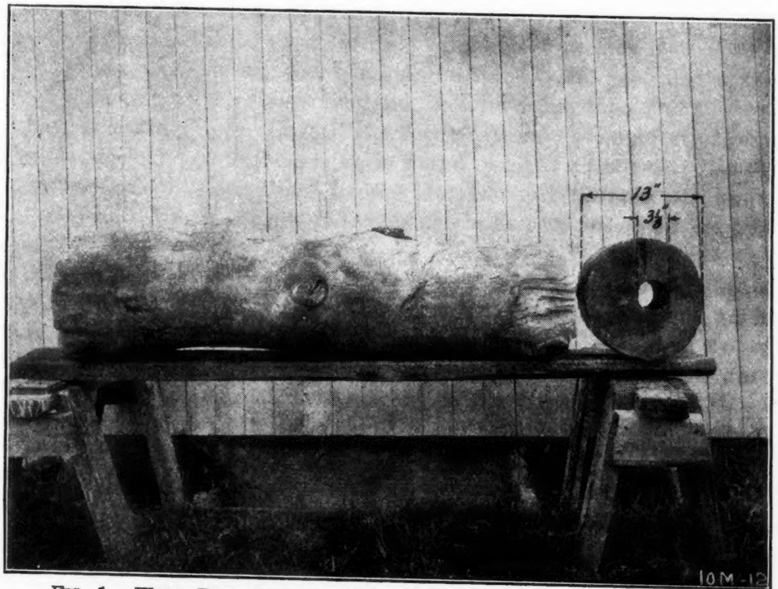

FIG. 1.—WOOD PIPE, ORIGINAL WATER SUPPLY LINE, HARTFORD, CONN.

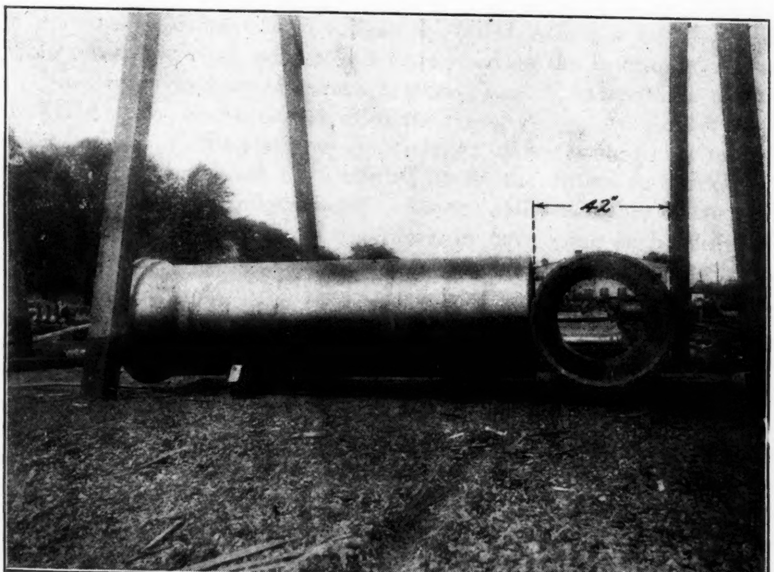

FIG. 2.—CAST-IRON PIPE, PRESENT WATER SUPPLY LINE, HARTFORD, CONN.

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photograph of one of these wooden pipes removed in 1926 in the center of the city during the construction of a large building. On Fig. 2 is shown a 42-in. cast-iron pipe for purposes of comparison.

The use of cement-lined pipes seems to deserve somewhat more attention than Mr. Fuller was able to accord it. As he states,* this pipe had considerable vogue, particularly in New England, and its value then and now seems not to have been fully appreciated. The best treatise on the subject of cement-lined water pipes known to the writer, is found in a paper† entitled "Wrought-Iron Cement-Lined Water Pipe" by the late Leonard Metcalf, M. Am. Soc. C. E. A perusal of this paper indicates the widest variation in experience, from good to bad.

As an example of satisfactory use, it is of interest to note the 30-in. cement-lined main of the old Charlestown, Mass., Water-Works, laid from the reservoir on Walnut Hill through Broadway, Somerville, Mass., for a distance of several miles. Installed in July, 1871, it is still in service as an integral part of the present Metropolitan (Boston) Water Supply System. Possibly, in systems where so much trouble was experienced with this kind of pipe, the makers and not the material were most to blame.

The reliance on linings and coatings to protect both cast-iron and steel pipes is mentioned very briefly by Mr. Fuller. The absolute dependence on the integrity of a covering to protect cast-iron pipes from rust tubercles that reduce flow and form corrosion that soon destroys steel pipe when exposed to "aggressive" waters is, however, worthy of more than a passing comment. The importance of the work of the committees of the several water-works associations now considering this matter of pipe covering, is realized when one recalls the steel pipes that have completely failed within a few years after being laid in adverse conditions of soil or exposed on the interior to corrosive water. Steel pipes undoubtedly have a large place in water-works construction, but that place must be suitable for their use.

In the matter of "distribution economics,"‡ it seems desirable to record the vast improvement that has resulted from the intensive work of the National Board of Fire Underwriters, under the able leadership of George W. Booth, M. Am. Soc. C. E., Chief Engineer, in developing such adequacy of supply and delivery systems that it has been possible to reduce rates for insurance considerably; which is only another way of stating that the fire hazard has been greatly diminished by the advancement in water supply work. Present conditions in a large Western city indicate that if one is searching for information concerning the "bitterness of the opposition to metering" it is unnecessary to revert to the Eighties.

The methods used in Philadelphia, Pa., and New York, N. Y., in securing a source of water supply were similar to those in Boston, when the first water supply was brought in through the "ancient conduit" installed by a company of citizens under authority of an Act of the May Session of the General Court of the Massachusetts Colony in 1652, or six years previous

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1590.

† *Journal*, New England Water Works Assoc., March, 1909, p. 1.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1591.

to the dug wells of the Dutch in 1658 and 141 years prior to the Manhattan Company of Aaron Burr, chartered by the New York Legislature in 1799. Two years earlier than that date, 1797, the Hartford Aqueduct Company was given a charter by the General Assembly of Connecticut. In February, 1795, the General Court of Massachusetts gave a charter to the "Jamaica Pond Aqueduct Corporation" to bring water from "any part of the Town of Roxbury into the Town of Boston."

In skimming the history of the development of pumping engines, the performance of the E. D. Leavitt triple expansion engine, installed at the Chestnut Hill Pumping Station of the Boston Water-Works in 1895 should not be overlooked. This pumping engine, on the authority of the late Desmond Fitzgerald, Past-President Am. Soc. C. E., is said to be "one of the most remarkable in existence in respect to duty and workmanship." Its three cylinders are 13.7 in., 24.375 in., and 39 in. in diameter, respectively. High speed was made possible by use of Riedler valve construction. The normal speed of the engine is 50 rev. per min., at which the pump capacity is 20 000 000 U. S. gal. per 24 hours. The duty of this engine was 145 000 000 ft-lb. per 1 000 000 B. t. u. It is stated that this engine has been run at a speed of 70 rev. per min., or 40% in excess of the contract, with smooth and economical operation.

The epoch-making Louisville, Ky., investigations and their result in directing future work in water purification, was due almost entirely to the minute attention to detail, the untiring enthusiasm and good judgment of Mr. Fuller.

It would seem desirable to call more attention to the two types of filtration works on which dependence is placed for safe-guarding public health and supplying attractive water; and especially to the advance made during the past 30 years in the art of clarifying the turbid waters of the South and West; due, first to the hit-or-miss methods of commercial exploitation and, finally, placed on a sound scientific basis by the technical work of the late E. B. Weston, M. Am. Soc. C. E., Mr. Fuller, and Robert Spurr Weston, M. Am. Soc. C. E.

The failure of the slow sand filter, in many cases, was due not to the type of filter, but to its limited application and inadaptability to meet some conditions imposed. To overcome these difficulties, modern art has developed a much more efficient machine than the slow sand filter. The largest slow sand plants in this country are those at Pittsburgh, Pa., Philadelphia, and Washington, D. C. Slow sand plants giving most satisfactory results as to quality of effluent and cost of production are in operation at Springfield, Mass., and at Hartford. In these places, as in many others in the north and east portions of the United States, where the natural waters carry little turbidity and only a moderate amount of color, the slow sand filter is particularly well adapted. Where this type can be used, the resulting effluent generally is much more satisfactory. After-sterilization is usually unnecessary and the operation of the plant is more "fool proof" than the rapid sand type. The latest of the larger filter plants of the rapid sand type is the installation at Providence, R. I., designed and constructed under advice of Allen Hazen, M. Am. Soc. C. E.

The rise in regulation of public utility works in the past two decades seems well worth comment and has demonstrated the wisdom of this method of control of water supply as well as other utility business. This regulation has injected into utility operation business methods which have been almost as efficacious in producing satisfactory working conditions as the improvement in technique has advanced the quality of the water supplied. In fact, these two are interdependent, for without economical financial control, the improvements in supply and purification would have been much less available for public use.

In a statement of this kind covering as it does a period of many years, it seems desirable to give brief note of some of the leaders in this branch of engineering who have now passed on. Among these may be mentioned James B. Francis, J. James R. Croes, Alphonse Fteley, Frederic P. Stearns, and Desmond FitzGerald, Past-Presidents, Am. Soc. C. E.; Hiram F. Mills, Hon. M. Am. Soc. C. E.; and Joseph P. Davis, Dexter Brackett, and George C. Whipple, Members, Am. Soc. C. E.

It is of interest to note also that most, if not all these names, as well as those of many others eminent in water supply work, were associated at times either with the Boston Water-Works, or that institute of public health work administered so efficiently by Dr. Thomas M. Drown, William T. Sedgwick, and Dr. Henry P. Walcott, the Massachusetts State Board of Health.

W. Kiersted,* M. Am. Soc. C. E. (by letter).†—Mr. Fuller's interesting historical sketch of water-works development reminds the writer of the situation in the Middle West in the Eighties and early Nineties, when promoters enthusiastically sought franchises for water-works construction. Due to their efforts, doubtless many small cities acquired water-works much sooner than would have been the case had the matter been left entirely to municipal initiative. The utility bonds offered by these promoters, for a time, seemed to have a ready sale, particularly for properties in the larger cities. However, with some of the promoters the essential aim was to increase their regular material and supply business through sales to the water-works constructor while accepting bonds in payment. The outcome of this policy, when the bonds issued against the utility failed to find a ready market, was sometimes disastrous and caused serious financial embarrassment.

Improvements were made from time to time as the method of purifying and distributing water became better understood, particularly in localities depending on a river as the source of supply. Mechanical filtration, one of the methods advanced through commercial channels as offering great prospects for improving the quality of river water, had its capabilities greatly overrated at first, which resulted in numerous failures in efforts to clarify muddy water. Through all this period a few important cities like St. Louis and Kansas City, Mo., and Omaha, Nebr., located on the Missouri River, continued to rely on sedimentation as a means of water clarification; and of these three cities, Kansas City is the last to construct what are termed modern purifica-

* Cons. Engr., Kansas City, Mo.

† Received by the Secretary, October 1, 1927.

tion works. Mr. Fuller could have consistently added a paragraph with regard to the evolution of the sedimentation process, particularly as it is the main dependence of successful filtration. The experience of Kansas City is a good illustration of what can be accomplished in clarifying water by sedimentation and of rendering a water safe and potable through the aid of chlorination. It is worth while to give the experience of this city during the fiscal years, 1921 to 1926. These general results are given in Table 5.

TABLE 5.—PHYSICAL CHARACTERISTICS OF KANSAS CITY, MO., WATER SUPPLY.

Fiscal year.	Average turbidity of water distributed, in parts per million.	Percentage of turbidity removed.	Typhoid deaths per 100 000.
1921	23	96.68	11.5
1922	23	98.28	4.6
1923	29	97.7	4.8
1924	16	95.5	3.6
1925	11	99.54	1.9
1926	8	99.68	3.46

In a large measure these highly gratifying results are due to the efficient effort of Dr. George F. Gilkison, although he was compelled to use settling basins not altogether scientifically designed. The results of sedimentation considered in connection with the low typhoid death rate seems to be conclusive evidence that the mechanical filter is, in fact, of secondary importance in clarifying and purifying the river waters of the Central West. Future development of the art of water purification should see more attention given to the design of the settling basin and to methods of breaking up the colloidal condition of sediment in the preparation of water for the filter. Further development of the filter itself should be directed toward simplifying and cheapening its construction. The topic of sedimentation is too important to be obscured by the prominence of an associated topic of less real importance, namely, that of mechanical filtration. The latter process must rely on the settling basin to prepare the water for filtration and on chlorination to sterilize the filtrate.

Mr. Fuller's suggestion* that the collection and digestion of available information relating to the engineering aspects of underground supplies, is a valuable one because this source of supply is greatly under-rated in localities where underground water supplies are abundant. Lack of an appreciation of the value of the underground source of water supply is perhaps the reason for advising cities, in a few instances, to abandon an available and abundant underground water supply for one from a polluted and muddy river. Propaganda relating to the mechanical filter may be responsible indirectly in no small measure for the obscurity surrounding the underground water supply in that it has engrossed, so completely, the minds of engineers.

It is worthy of note, however, that late developments show that a large quantity of water can be pumped from a single well sunk in fine water-bearing sand. This is certainly an achievement, particularly in the method by

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1594.

which wells in such material can be successfully constructed, operated, and maintained. Success in this particular direction has greatly broadened the range of development of underground water and has added a new stimulus to further effort along similar lines.

Referring to Mr. Eddy's paper the developing stage through which the disposal of sewage is now passing is quite likely to leave in its wake many abandoned or transformed structures formerly erected in compliance with some theory or principle which, at the time, appeared promising. The septic tank is now of the past for community use. The Imhoff tank, although still a recognized device for treating sewage, will likewise become a thing of the past, except for small towns, because experience has shown the insurmountable difficulties of attempting to combine, successfully, the purely mechanical process of sedimentation with the biological process of sludge digestion. A sacrifice of essential principles of one or the other of the processes is likely to follow attempts of co-ordinating them in a single structure.

Preliminary sedimentation is an essential factor for consideration in any method of sewage purification, and its importance is so great as to entitle it to an independent structure wherein the governing principles of sedimentation can be applied without interference with those of an entirely different process. This treatment cannot accomplish all that is desired, however, because it does not remove the colloids and, therefore, does not reduce the sewage to a condition suitable for application to either land or sprinkling filter. The desirable intermediate treatment is one that will break up the colloidal condition of the remaining sediment and reduce the sewage to a state susceptible of further improvement in a secondary settling basin. The writer regards this intermediate treatment of sewage as the most important phase. It invites the closest attention of the chemist and the engineer in arriving at the best and most economical process by which a maximum percentage of suspended matter can be removed from sewage before attempting to purify it still further by any natural process. When this end is accomplished the final disposal of sewage by irrigation in the arid and semi-arid sections of the United States, or by intermittent filtration in other sections of the country more generously watered by Nature, will receive well merited consideration. Neither method of land disposal should be considered obsolete as long as a natural process of sewage purification remains acceptable.

The sprinkling filter, notwithstanding its many merits, must disgorge itself of organic accumulations periodically or become clogged. The usual offensiveness of the disgorging process can be alleviated in a great measure through the previously mentioned intermediate treatment. Then the danger of overburdening the natural activities will be decreased; there will be less occasion for the use of after-sedimentation; and the area of the filter, always expensive to construct, can be materially reduced.

The method of sludge disposal, which is of secondary importance, should be adapted, of course, to local surroundings. Although they are accompanied with some odor, reservoirs cheaply constructed of earth offer a satisfactory

solution in isolated places. In other localities, open or closed separate digestion tanks may be required, wherein some kind of treatment will probably be needed.

In short, it is the writer's belief that future development should be directed toward a larger removal of suspended matter before sewage is applied to land or sprinkling filters.

M. M. O'SHAUGHNESSY,* M. AM. SOC. C. E. (by letter).†—Mr. Fuller is to be complimented on the exhaustive handling of the subject of water-works, included in his brief summary. The most valuable tabulation is the great increase in number of publicly owned water systems *versus* privately owned (Table 1‡). The United States is to be congratulated on making the greatest advance in the science of water distribution and measurement as indicated by the Venturi meter developed by Clemens Herschel, Past-President, Am. Soc. C. E.

In describing hydraulic-fill dams§ Mr. Fuller ignored the credit that belongs to the late Julius M. Howells, M. Am. Soc. C. E. He successfully constructed the first two hydraulic-fill dams in the United States; the first in Texas and the second at La Mesa, San Diego, about 70 ft. high. The latter has since been submerged by a higher buttressed arch concrete dam immediately below it. The late James D. Schuyler, M. Am. Soc. C. E., had the discretion to pick up Mr. Howells' ideas and methods and, with a broader practice, develop the subject of hydraulic-fill dams more exhaustively.

Mr. Fuller ignores the importance of the rock-fill dam, devoting less than one sentence to it. It is entitled to more consideration, as the largest dam east of the Mississippi River is the rock-fill dam, more than 280 ft. high, across the Dix River in Kentucky, which was completed in 1925 as a restraining reservoir for power uses. The water-tight curtain is properly on the water face of the dam where it belongs, and not on the back.

He contributes a very valuable section toward the history of pumping water, including a description of the Worthington pumps. Modern electrically operated centrifugal pumps have displaced all the heavy, cumbersome machinery and equipment in vogue thirty years ago.

The Medical and Engineering Professions are to be complimented on the improved sanitary condition of water supplies for drinking purposes, as indicated by the lowering typhoid fever death rates in American cities. Table 2‖ shows at once that there is at present only one-twentieth of the deaths from this disease that there were 20 years ago. This shows progress.

The City of San Francisco has taken elaborate precautions for the preservation and purification of its water supply by extensive land ownership around reservoirs. This is a wise precaution. It is a foolish idea for water companies and cities to encourage pleasure-seeking people to enjoy the fishing

* City Engr., San Francisco, Calif.

† Received by the Secretary, October 4, 1927.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1588.

§ *Loc. cit.*, p. 1953.

‖ *Loc. cit.*, pp. 1598–1599.

or boating on domestic water supplies, and the only safe rule for public health is exclusion. In this regulation the San Francisco engineers are supported by the public health authorities.

JOHN H. GREGORY,* M. AM. SOC. C. E.—Mr. Eddy has presented an exceedingly interesting paper on the history of the sewerage and drainage of towns. There are many phases of the subject that might be discussed at length, but the speaker will confine his discussion to one phase only; that is, the question of the separate or combined system of sewers for cities. There is much to be said for both systems. The speaker holds no brief for either type, but there are some phases of the subject that should be considered rather carefully. Take, for example, the question of storm overflows from a combined system. When a large city is located on a stream which has a large volume of flow, the question of storm overflows may not be of much importance; whereas, if the city is located on a stream the flow of which during dry weather may be almost nothing, storm overflows become of great importance.

With reference to maintenance and operation, both systems present certain difficulties. With the combined system there are catch-basins and, with the separate system, surreptitious connections of rain-water leaders to the sanitary sewers, sometimes causing flooding of cellars, and also surreptitious connections whereby sewage is discharged into storm-water drains.

However, it seems that there is very much to be said for the separate system in the case of a large city located on a stream that has a very low stream flow in summer, especially if the stream runs through the heart of the city. The speaker has in mind one city having a population of about 300 000 which is sewered mainly on the combined system. The city happens to be located on a stream having a water-shed of 1 500 sq. miles and a dry-weather flow in the stream of less than 10 cu. ft. per sec.; which is almost nothing. Under such conditions storm-water overflows are of great importance, and the volume of overflow must be kept at a minimum if nuisance in the stream is to be avoided. If the city was sewered on the separate system, the problem of keeping the stream clean would be much simpler.

The tendency in the country is for cleaner streams, and if streams are to be kept clean it is a question whether the advantages do not lie with the separate system, rather than with the combined system, if the sewage is to be treated.

KARL IMHOFF,† M. AM. SOC. C. E.—The scientific conditions of the digestion of sewage sludge in separate tanks (separated from the flowing sewage) were first studied by Mr. H. W. Clark, in Boston, Mass., in 1899. Since 1906 sludge digestion has been successfully carried out in the lower chamber of two-story settling and sludge-digesting tanks. Later, sanitary engineers learned also to digest sludge successfully in entirely separate tanks by the same alkaline methane fermentation that is observed in the lower chamber of good two-story tanks.

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† Chf. Engr., "Ruhr-Verband," Ruhrverband, Essen, Germany.

The digestion depends only on two factors: Mixing and temperature. The fresh sludge is mixed with ripe alkaline sludge in order to avoid acid fermentation in the former. The pH value of ripe sludge is 7.3 to 7.6.

The temperature should be between 6° and 25° cent., the higher the better. In the Southern States the natural temperature of separate tanks may be sufficient. In the Northern States, two-story tanks are better because the sludge tank is kept warm by the flowing sewage of the upper chamber. Artificial heating is easier in separate tanks.

Liming (against acid fermentation) was first tried in 1913 by W. L. Stevenson, M. Am. Soc. C. E., and K. Thumm, in Berlin. It is also recommended by Dr. W. Rudolfs. It may have value in ripening time, or for special acid trade waste; but, ordinarily, it is not necessary.

Kenneth Allen,* M. Am. Soc. C. E. (by letter).†—Mr. Eddy has presented an adequate and comprehensive review of progress made in salient features of sewerage and sewage disposal practice in a few pages of print to which little of importance can be added. Nevertheless, the following notes may be of enough interest to be mentioned.

The separate system of sewers was advocated by Edwin Chadwick in England in 1842 and by John Phillips in 1847, more than thirty years before its introduction in the United States by the late Col. George E. Waring. The City of Paris built its first modern sewer in the rue de Rivoli in 1851. The practice of flushing to remove deposits in the crudely designed sewers of the period by damming and then releasing the flow had been used to a considerable extent in London, England, and in Germany; but with the advent of the separate system, in which deposits were liable to occur because of the small diameters used and the intermittent flows of sewage near the upper ends of laterals, the Rogers-Field automatic flush tank came into use as a more practicable device. These tanks were deemed an essential accessory to such systems by Waring, who introduced them at Memphis, Tenn., and, with later modifications, they were largely adopted in the design of separate systems during the latter part of the Nineteenth Century, until it became apparent that their effect was more limited and their use of water more costly than at first supposed. They still find a legitimate field of usefulness under restricted conditions.

Concerning the introduction of sewerage in New York, it may be remarked that the first "Public Necessity House", or comfort station, was built in 1691, while the first public sewer is said to have been laid in Broad Street, as a result of a petition to the Town Council, in 1696; but it probably was not until 1805-07 that an important sewer, still incorporated in the existing system, was built in Canal Street. In 1860 egg-shaped sewers were introduced and, in 1864, vitrified pipe.

The first sewer in Chicago, Ill., is said to have been laid in 1835, but it was not until 1856 that construction under a systematic plan was put into effect.

* San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

† Received by the Secretary, October 29, 1927.

It is said that water-closets existed in Herculaneum in 400 B. C., but in spite of the precedent it was not until 1776 that a patent was issued for this device. In 1846, plumbing was introduced in America as a special luxury. Siphon traps were suggested for water-closets by the Metropolitan Sanitary Commission in England in 1847 and by 1852 they were being introduced in that country. A quarter of a century intervened, however, before the use of water-closets became common in the United States.

The question of sewer ventilation has received more attention in England and on the Continent than in this country, where reliance has been usually placed on holes in the manhole covers. These were introduced in New York in 1875. Where the sewers are designed or operated so that they keep clean, this seems to meet the sanitary demands of the case. It was just before this (1873) that Baldwin Latham published his "Sanitary Engineering", which is probably the first comprehensive treatise on the subject of sewerage in the English language.

Mr. Eddy has referred to the screw pumps at Milwaukee, Wis., Chicago, Ill., and New Orleans, La.* A similar pump was installed about fifteen years ago at the head of Gowanus Canal, Brooklyn, N. Y., to flush the waters of that stream by pumping, either from it into a 12-ft. tunnel, 6 270 ft. long, leading to Buttermilk Channel, or *vice versa*. The capacity of the pump is 14 000 000 gal. per hour.

The practice of discharging sewage or sewage effluents from submerged outlets has met with increasing favor since the construction of this type at Hamburg, Germany, Boston, Mass., and Washington, D. C., about 1900. The advantage is very apparent where the depth is more than 30 ft. and where, as in more recent construction at Cleveland, Ohio, Deer Island (Boston), and in the case of the Passaic Valley sewage, discharge is distributed by means of multiple outlets.

In 1647, an Act to Prevent Pollution of Boston Harbor was passed, but for more than two and a half centuries nothing of importance was done to preserve the purity of American rivers, lakes, and harbors.

In 1876, the Massachusetts State Board of Health issued a special report dealing with the "Pollution of Rivers", the "Water Supply, Drainage, and Sewerage of the State", and on the "Disposal of Sewage", calling attention to methods used abroad and to the necessity of adopting similar measures in the interests of sanitation. This, and a report the following year, on "Pollution of Streams", were notable in rendering the results of foreign experience available to American sanitarians and engineers.

The first comprehensive report on methods of disposal, presenting definite recommendations for construction, was submitted by the late Samuel M. Gray, M. Am. Soc. C. E., to the City Council of Providence, R. I., in 1884, although a few attempts at disposal on a small scale had already been made. The first attempt, involving irrigation, had been made at an insane asylum at Augusta, Me., in 1876, and at Cheyenne, Wyo., in 1883.

The water-closet was by that time being introduced and the condition of watercourses in the more densely populated districts was becoming more

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1605.

serious. Land disposal was in vogue as a method, and chemical precipitation was coming into favor. The plant recommended by Mr. Gray was built at Providence and was followed by others elsewhere, as mentioned by Mr. Eddy. Among the smaller plants was that utilizing the Dortmund type of tank designed by Allen Hazen, M. Am. Soc. C. E., and operated at the World's Columbian Exposition in Chicago, in 1893.

During the two decades following 1880, irrigation or intermittent filtration was introduced at Pullman, Ill., South Framingham, and Brockton, Mass., Plainfield, N. J., Pawtucket, R. I., Canton, Ohio, and elsewhere. The experiments on filtration, conducted by the Massachusetts State Board of Health, at Lawrence, during and subsequent to the late Nineties, under the direction of the late Hiram F. Mills, Hon. M. Am. Soc. C. E., added emphasis to the important part played by air, and were followed by the development of the contact bed and trickling filter. At Reading, Pa., a double-deck filter was constructed above ground, where screened sewage was first strained through a bed of coke and delivered to a filter of sand resting on cinders. After percolating through this bed the sewage fell about 10 ft., in the form of rain, to a second filter consisting of sand resting on slag. The process provided a good effluent, but proved expensive and, later, was abandoned.

In 1909, aerators, leading the air caught by cowls to the bottom of trickling filters, were introduced by the late Rudolph Hering, M. Am. Soc. C. E., at Atlanta, Ga.

Among the more recent applications of air to filtration should be mentioned that of Strogonoff, at Moscow, where oxidation is hastened by introducing air in the bottom of an enclosed filter of fine material about 11 ft. deep. In this way it is claimed that ten times the volume of sewage can be treated per acre as by the ordinary trickling filter.

Meanwhile, the septic tank came into favor for sedimentation, followed by the Imhoff tank, of which type the Calumet Plant at Chicago deserves mention, while methods of artificial aeration were being attempted in a small way.

The important rôle played by the free contact with air in the treatment of sewage was recognized in England by Angus Smith as early as 1882. In 1891, Col. Waring began experiments at Newport, R. I., by forcing air into a bed of gravel or broken stone, which process he patented and introduced at small plants in Brooklyn, N. Y., Philadelphia, and Wayne, Pa., and elsewhere. In 1892, Lowcock came out with a somewhat similar process in England. In 1910, Black and Phelps conducted experiments in forcing air through colloider tanks at the 26th Ward Plant, Brooklyn, and, in 1913-14, the activated sludge process was developed by Fowler and his associates at Manchester, England. It was patented in America in 1915 by Mr. Leslie C. Frank.

A recent development along the lines of activated sludge has been made by Karl Imhoff, M. Am. Soc. C. E., in what is called an "Emscher" filter, or "contact aerator", introduced at Kettwig, Germany, in 1925. In this a certain degree of activation is brought about through the artificial aeration of the biological growth on filter media submerged in the upper compartment of an Imhoff tank, resulting in the elimination of separate aeration tanks.

Disinfection of sewage by calcium hypochlorite was tried out in 1884 in London by Dibdin and was investigated in Germany by Proskauer, Elsner, and Dunbar, and in the United States by Phelps, who, in 1907, applied it on a practical scale at Red Bank, N. J. More recently liquid chlorine has largely supplanted hypochlorite, and its use has extended rapidly for the protection of shellfish and bathing beaches. It has also been advocated to defer the oxygen demand of effluents.

Reviewing the development of sewerage and sewage treatment in America, experience, first in England and, later, in Germany, has been freely utilized; but it has been modified to the different climatic demands and in accord with independent research, of which the outstanding example was the early work of the Massachusetts State Board of Health. Irrigation, intermittent filtration, chemical precipitation, the contact filter, and the septic tank, have all seen their day; while fine screening, the Imhoff tank, sedimentation tanks with mechanical sludge removal, the activated sludge process, and chlorination, hold their own as standard and reliable methods of treatment.

L. L. Tribus,* M. Am. Soc. C. E.—Mr. Greeley's paper relating 150 years of municipal history, has naturally included many items of interest, but of necessity has had to omit enlargement where it sometimes would have proved very interesting.

No one can remember conditions farther back than perhaps one-third of the period, but the important phases of municipal housekeeping that comprise the cleaning of streets, gathering of waste products, and disposing of them, have all been developed in the past 50 years.

The perfection of pavement cleanliness now demanded, was impossible (even if wanted), prior to the advent of hard and smooth pavements. Macadam could not be machine-swept without risk of injury to the surface coat, unless maintained with greater care and at larger cost than cities would sanction. Cobble-stones, which covered so many miles of some city streets, afforded ideal gathering places for innumerable filth germs.

So far as the speaker recalls, the only place on earth where cobble pavements had any real excuse for existence is at Funchal, in the Madeira Islands, where transportation is by ox teams drawing sleds with well-greased wooden runners.

As people advance in desires, their demand is for larger municipal operations; and *vice versa,* larger operations re-act on the people. The speaker well remembers certain experiments of about 20 years ago. He selected three ill-paved, ill-kept, slum streets in different parts of a city. The buildings were largely devoid of paint; windows were rarely fully glazed; shutters hung by one hinge; house refuse was thrown in the gutters; children abounded and, equally with their parents, were a slovenly lot.

The curbing was then straightened; good curb-to-curb pavements were laid; a street sweeper in white uniform was assigned to each street; and notices were served asking house occupants to put refuse in sound receptacles, for regular daily collection by the city. What happened? Within six months windows

* Cons. Engr. (Tribus & Massa), New York, N. Y.

were glazed; shutters hung; painting had begun; sidewalks were being repaired or renewed; garbage and wastes were almost completely placed in metal containers; some plate-glass store fronts appeared; children were less tattered and more frequently actually washed; and the parents began to be self-respecting. In one year the transformation was complete, and has been lasting.

Mr. Greeley has called attention* to the basic fact in municipal housecleaning problems, that only when those at the head take a real interest in their jobs, does efficiency show in the work itself. An enthusiastic head, honest and capable, will secure, out of even poor raw material, results that will bring credit and community praise.

The inter-relationship of pavements and street cleanliness is so vital that only with the former in first-class shape can the latter obtain. Street cleanliness usually re-acts on the maintenance of properties, but not always.

Municipalizing garbage and refuse collection and treatment systems is rather the last great step, aside from transportation, in city life. Waterworks, sewage systems, and various other utilities reach approximate perfection, but private ash collection continues, with its greater per capita cost, until places for dumping refuse or for feeding garbage to hogs, and for reduction processes, cannot be readily obtained. Then municipal relief is demanded.

The Oriental and near-Oriental sensibility to smell and sight does not seem as keen as that of the conglomerate and amalgamated American with its Anglo-Saxon predominance; so the dog and vulture scavengers that comprise the street-cleaning personnel of many populous cities are even yet tolerated.

The speaker is glad that he visited Constantinople before the thousands of street dogs were destroyed, and saw the groups of mangy curs, large and small, appropriating the sidewalks, while pedestrians walked in the streets. These groups knew their own districts, perhaps by virtue of dog vote and determination, for it was immediate destruction for any dog to step outside his own territory. Not a scrap of any edible refuse could long be found in the streets of that wonderful cosmopolitan city. The unpaid, non-uniformed, four-footed corps did the trick, until finally banished to a neighboring island. They are said to have disappeared through the process of killing and eating each other. Lack of general sanitation, however, produces a terrible death toll when epidemic breaks out in Oriental cities.

Mr. Greeley has outlined the types of treatment plants with some of their respective merits, but on the economic side has not had time to emphasize sufficiently two points: (*a*) The relationship of costs of collection and haul, to the locations, sizes, and types of disposal systems; and (*b*) the possible market, if any, and means of reaching it for recoveries of by-products and residues.

As a general principle, a large community must consider the whole services as one of net expense, with perhaps opportunity for some returns by the way.

For historical accuracy as to the introduction of the English furnace into this country, the speaker desires to give large credit to Mr. George Cromwell, who as President of the Borough of Richmond, City of New York, by his broad-gauged intelligent co-operation and authorizations permitted the completion of a long course of experiments. These were conducted by the

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1620.

writer, with Mr. Richard T. Fox (later of Chicago, Ill.), and John T. Fetherston, M. Am. Soc. C. E. (later, Commissioner of Street Cleaning of the three larger boroughs of New York), as chief assistants.

The findings were confirmed by Mr. Fetherston in a visit to Great Britain and the English type of high-temperature incinerator was subsequently adopted to serve the needs of Staten Island.

The speaker's special contribution to the change from the original English type of furnace, was the joint cooling of the clinker and heating of the forced-draft air, by driving it through the hot clinker while held on a lower grate. Mr. Fetherston's very valuable and useful patent was the making of a ridged grate, which formed breakage lines in the clinker, rendering its removal simple. The hydraulic charging pan was first perfected in the West New Brighton, Staten Island, N. Y., station.

Concurrently, with the Staten Island experiments and independently therefrom, the City Engineer of Seattle, Wash., reached the same conclusion, namely, that high-temperature incineration was the best form where utilization was not expedient and where the early destruction of organic matters was essential.

Staten Island seems to have been, not only the early nursery of scientific garbage treatment, following its first two Dixon furnaces, but the site of a very large Cobwell reduction plant that furnished cause for much litigation, throughout the course of which the speaker was consultant to those in opposition, led by the District Attorney and committees of the people. Briefly, the proven excessive nuisance from odors caused the final closing of the plant, not condemning the Cobwell system *per se,* but rather the operation of that particular installation.

Smooth, hard pavements, rubber tires, and horseless vehicles have lessened street noise and pavement wear, making street cleaning much simpler, more efficient, and cheaper than formerly. Municipal garbage and refuse collection, with treatment in high-temperature furnaces, has made for betterment.

Snow removal is a factor in large city housekeeping in the temperate zones. "Old Sol" looks after the situation in the southern range of cities where snow occasionally falls. "Jack Frost" largely holds the snow in place in the northern communities, until spring thawing attends to the matter; sidewalks are cleared, adding to the depth of snow in the roadways where sleigh and sled traffic soon packs it down.

More of the largely populated places come within the first series, and the problems have been many. Gathering by scrapers, and hand-shoveling, with carting to water-front dumps, have been most generally adopted; but probably the most efficient method, where sewers are large enough, is gathering and dumping into sewer manholes, aided, if the weather be not too cold, with urge of water from fire hose lines. A light fall may even be wholly flushed by hose into corner basins. The total of annual expense runs into many millions of dollars.

What the next 150 years may bring forth is hard to predict. Perhaps boneless synthetic foods will be prepared that will entirely eliminate garbage

wastes, and only the containers will require disposal! The imagination might well run riot, as to the future, in view of past achievements—so well reviewed by Mr. Greeley. In general, however, engineers had better report upon accomplishment, rather than prophesy.

ARTHUR E. MORGAN,* M. AM. SOC. C. E.—In every phase of the development outlined by Mr. McCrory is seen the same characteristic of gradual growth from crude and simple origins to great and complicated operations. The author describes the beginning of tile drainage in New York State 50 years ago with the use of 1, 2, and 3-in. tiles, and the beginning of open drainage with small farm ditches. From these have developed the extensive systems of underground drains in the Middle West, laid with great tile 2 or 3 ft. in diameter, and the open channels, sometimes 100 ft., or more, in width, in systems draining hundreds of square miles. In the development of excavating machinery there has been the same gradual growth, from the time when all work was done by hand, to the present day of dipper dredges, drag-lines, cableway excavators, and suction dredges.

In the associated legal developments the same trend from the old primitive laws, enabling a few farmers to co-operate, each digging his share of the neighborhood ditch, to some of the modern codes, under which as many as 50 000 tracts of land have been assessed to pay the cost of a single improvement.

In most of that legal development it has been characteristic that lawmakers have seen but a little way ahead. The speaker has been interested in following the history of drainage law. As a rule each statute that has been passed, has been in the interest of some single project or type of project, and reflects the particular experiences of those who drafted it. Each law generally accepts the code that has gone before, and makes modifications of particular features. The process has been a step-by-step development, each new code carrying along the vestiges and appendages of obsolete legal and administrative procedure. Land drainage engineers are only beginning to devise and create codes that are new in their general structure, based on comprehensive study of needs and of legal and administrative procedure.

The progress of drainage has depended, not upon the improvement of any single factor, but upon the co-ordinate development of all factors, technical, mechanical, legal, and administrative, that enter into such a development. If a single factor falls behind, the whole development must suffer.

Some years ago the speaker saw a drainage canal recently excavated by a dipper dredge to drain part of the Dismal Swamp in Virginia. The work was done from plans made by George Washington. In his early days he had dreamed of draining the Dismal Swamp, but his plans waited 150 years because no equipment was available for their execution. The speaker first worked with Mr. McCrory in 1907, in Colorado. There they made plans for reclaiming "alkali" lands which, if executed, would reclaim a large waste area; but because of inadequate legislation 15 years passed before any substantial progress was made.

* Pres., Dayton Morgan Eng. Co.; Pres., Antioch Coll., Dayton, Ohio.

The process of reclamation does not depend on engineering design or on excavating equipment alone, but on the co-ordinate development of every factor that is involved. In his proper function the engineer is the person who co-ordinates and synchronizes all these factors. Anything that affects the ultimate success of the project is a proper part of his work. Sometimes, the engineer sees himself as concerned only with technical design. Until he can rise above that concept, and can see his work to include all factors necessary for the proper development and co-ordination of all elements, he will not come into his own.

When the speaker first planned extensive drainage work in Minnesota before 1905, the drainage laws were very inadequate. He spent a year in revising the Drainage Code and in getting his revision approved by engineers and lawyers, and passed by the Legislature. There was no suitable power equipment in the State and there were no experienced contractors to operate the equipment; so he searched the entire country for both. In his opinion, these efforts were normal and proper elements of engineering.

In the subsequent 20 years of practice this conviction has been strengthened. Is it not true that the factor most lacking to-day in the control of water is a larger concept of his functions by the engineer? There are great projects in this country that never can be fully realized until the engineer has a picture of the possibilities, and until he possesses the co-ordinating mind that brings together all necessary factors.

The engineer must see himself as the center of these developments. He must be enough of a lawyer to appraise the legal factors; enough of an engineer to plan the destinies of streams rather than the expediency of particular cases; enough of a business man to co-ordinate and define policies; enough of a developer to bring about the necessary adaptation of mechanical equipment; and enough of an administrator to bring all these factors together to effect his purposes.

The time is coming when the co-ordinating work of the drainage and reclamation engineer must extend beyond State lines. On the Colorado River there is a deadlock because of the lack of interstate machinery. There are great projects in Missouri and Arkansas similarly waiting for the technique of interstate co-operation. The compact now being worked out between New York, Pennsylvania, and New Jersey to give unified control to the Delaware River is a step in the right direction.

The control of water should not be exercised with reference to any one interest, but all probable uses and effects of water must be considered in engineering programs. About 1915 the speaker endeavored to outline, for the Water Supply Division of the State of Pennsylvania, a statement of policy for the use of its waters for all purposes.* An effort was made to consider the water supply of the State as one of its major resources, to be administered for all its uses by a department of the State Government, that should pass upon conflicts of interest.

Water is one of the prime essentials of civilization, and must be controlled and used with recognition of all the interests involved, and not of one alone.

* Report of 1916.

Sanitation, agriculture, power development, navigation, fisheries, or water supply must not be considered alone, but each one in relation to all the others. Organization and design must replace the present anarchy.

A striking example is the control and use of the waters of the Great Lakes. Navigation interests assume that they are all-important; power interests endeavor to direct events to further their own ends; the City of Chicago thinks chiefly of the Great Lakes as furnishing water for waste disposal; and public sentiment inclines to the attitude that the flow from the Lakes should first of all be conserved to maintain scenic effects at Niagara. The Great Lakes present a single problem in the control and use of water. To realize that, a single co-ordinated system of control, with all interests and uses taken into account, must replace the present conflict of interests.

To bring about such changes as this, the development of engineering statesmanship is needed. The engineer, more than any one else, is able to see the whole problem, and to bring the comprehensive control of waters to a level with the best phases of present civilization. Just as efforts for the control of water have grown in importance from the farmer's ditch to the control of the Great Lakes, so the work of the hydraulic engineer must lead from the country drain surveyor to the engineering statesman who devises and administers the control and use of great water resources.

ARTHUR M. SHAW,* M. AM. SOC. C. E. (by letter).†—This paper presents, in an interesting manner, the evolution of the modern system of drainage, especially sub-drainage, as an adjunct to agricultural development. Few people who live in cities realize the vital importance of controlling soil moisture by drainage. Irrigation, being frequently more spectacular, appeals to the imagination although infinitely more agricultural products come from lands that have been drained artificially (to a greater or less extent) than are produced by irrigated lands.

The various steps in drainage became synchronized with the growth of the country, although it is doubtful whether artificial drainage ever was practiced to any great extent by the early New England colonists. With important exceptions, the practice of sub-drainage, originating in England, jumped the more rugged lands of New England and came to its greatest early development in Western New York. From there, the idea was taken into the "Western Reserve" and, later, into Indiana, Illinois, Iowa, and farther west and north.

By the time that these newer areas were brought under cultivation, it had been found that clay drain tile were so superior to the pioneer "blind drain", made of stones or brush, that few of the latter were used; but some temporary substitute was found necessary to serve the sections where tile could not be secured or where the farmers were not able to bear the initial expense. Such substitutes were the "Mole" and the "Bull Ditcher."

The original "mole" was a crude but heavily constructed sled with a steel plate extending down from the bottom of the sled to a depth of about 30 in. The plate was placed in such a position that, in pulling the sled along, a

* Cons. Engr., New Orleans, La.

† Received by the Secretary, October 14, 1927.

slit was cut in the ground by the edge of the plate, the lower end being given a sufficient "lead" to insure its remaining in the ground to its full depth. At the rear edge of the plate, near its bottom, was attached a short piece of chain, at the other end of which was a 4-in. forged iron ball; or, if available, an old cannon ball. The "machine" required about twenty yoke of oxen to pull it, although it frequently was pulled by one team of horses operating a home-made windlass.

The "bull ditcher" resembled a giant, double mold-board plow and was used in digging open ditches. In its later form, it could dig a ditch of 3 ft., or more, in depth. To avoid excessive draft, it usually was designed to construct a V-shaped ditch with side slopes of 2 on 1, or flatter. This equipment was used in the drainage of thousands of acres of prairie and marsh lands, principally in Northern Iowa and in Minnesota. The writer used an adaptation of the idea a few years ago in the preliminary ditching of a 1 500-acre tract of Louisiana "trembling prairie". A huge cypress log was sharpened at one end and provided with a stiff coulter and a pair of heavy plank wings for spreading the dirt. This crude appliance was pulled through the soft muck by a cable running to a donkey engine which was mounted on a barge. Canals had been dug at ¼-mile intervals and these served for transferring the power equipment.

Except under very favorable soil conditions, the useful life of the "mole" drain did not extend over a period of more than two or three years, but the "bull ditcher" proved to be a real success for its time. In digging comparatively shallow, flat slope ditches in suitable soil, free from rocks or stumps, it is doubtful if any modern machine could compete with it in its low cost of operation and upkeep.

A history of land drainage in the United States would not be complete without reference to the thousands of miles of open ditches still used in the drainage of the sugar plantations of Louisiana. Until recently they were dug and maintained by hand labor and this method is still utilized on the majority of the plantations, although machine ditchers of the wheel, or the endless chain and bucket type, are coming into use. Many experiments have been made in the sugar belt with tile sub-drains, but they have not been as successful as in other sections.

W. B. Gregory,* M. Am. Soc. C. E. (by letter).†—The paper by Mr. McCrory presents the main points in the history of modern drainage and, of necessity, the author has omitted many points of local interest.

To the inhabitants of the State of Louisiana the problem of drainage will always claim a lively interest. This State ranks second in the area of swamp lands, the State of Florida alone having a greater area. The City of New Orleans, the metropolis of the South, is built on alluvial land, a portion of which is below mean Gulf level. Except for narrow strips of land along the Mississippi River, the lands of Southeast Louisiana are at or near the level of the waters of the Gulf.

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† Received by the Secretary, October 28, 1927.

Under these conditions the problem of drainage by pumps becomes of considerable importance. The sugar plantations, with high lands near the river, were first drained by means of open ditches. Later, the amount of cultivable land was extended by building levees around the rear of the plantations and pumping out the drainage water. The pumps first used (as noted in the paper),* were drainage wheels, modeled after scoop wheels, used hundreds of years ago in Spain and other parts of Europe. In its later development these wheels were 28 to 30 ft. in diameter, with widths of from 6 to 8 ft. They were used to pump large volumes of water through small lifts, usually not more than one-fourth the diameter of the wheel.

One difficulty encountered with the scoop wheel was in the foundations, which were expensive and difficult to maintain. With the development of centrifugal pumps, the Ivens or the Menge pump was used because of the lower cost of installation, but not because of more efficient operation.

In turn, the centrifugal pump has found a competitor in the screw pump. The City of New Orleans is an example of the change in methods of pumping. The first pumps were of the scoop type, then the centrifugal, and now screw pumps are largely used.

Practically the same changes have taken place in the pumping of drainage water from agricultural lands, for while a few drainage wheels are still to be found on plantations, there are many more centrifugal pumps and some modern installations use screw pumps.

A notable example of drainage of swamp lands is to be found in the Fourth Jefferson Drainage District, comprising about 30 000 acres of Jefferson Parish, just west of the Parish of Orleans—the City of New Orleans. A large part of the area is swamp land, but the run-off from the high lands along the river is also to be pumped by four large pumping plants, each having two units of screw pumps, each of which is capable of removing approximately 120 000 gal. per min., at low lifts, the capacity falling off slightly at higher lifts. Each pump is driven by a 330-h.p., 4-cycle, Diesel engine. The net capacity of the pumps is sufficient to remove a run-off of 1.5 in. in 24 hours.

The drainage of swamp lands is highly desirable in Southern Louisiana for many reasons, among which may be mentioned the elimination of mosquito-breeding areas and the additional area of very fertile lands to be used as homes. There was an ambitious beginning of this work in the first decade of the Twentieth Century, but the work was halted by the World War and its attending economic conditions. With vast areas of uplands in this section, that may be drained by gravity and yet are uncultivated (although not as fertile as alluvial lands), there is little incentive at present to drain swamp lands unless they are near enough to a city to promise returns that are not of a purely agricultural nature. However, some economists are discussing the approach of an era when the production of food for a vastly increasing population will be one of the gigantic problems. When the time comes that an agricultural return will justify the outlay for swamp-land reclamation, this

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1632.

State and many other sections of the South will come into its own and will drain these wet lands which are among the most fertile and productive to be found in the world.

H. F. GRAY,* M. AM. SOC. C. E. (by letter).†—At the beginning of his paragraph on "The Public and a New Idea",‡ Mr. LePrince states that no attempt was made to control disease-bearing mosquitoes in the United States until 1913.

Pioneer work was done in California as early as 1910, and has continued since that time.§ The initiative was taken by William B. Herms, now Professor of Parasitology, College of Agriculture, University of California. Under his direction, what was probably the first campaign in the United States for malaria control through the prevention of mosquito breeding was undertaken at Penryn, Calif., beginning early in March, 1910. Late in the same month, another campaign for the same purpose was begun at Oroville, Calif. In August, 1910, Bakersfield, Calif., began an anti-mosquito campaign. The first malaria-control campaign on an irrigation district was carried on at Los Molinos, Calif., in 1912, as related‖ by Thomas H. Means, M. Am. Soc. C. E.

Although these early campaigns were intended primarily as practical demonstrations of the possibility of malaria control through mosquito reduction, were handicapped by inadequate funds, and lacked official recognition through statute authority, they were so successful that the work was continued under various auspices, and extended to many other communities in the State. Finally, in 1915, the California Legislature passed a bill¶ authorizing the formation of mosquito-abatement districts, and providing for funds through taxation of the lands benefited. At present, there are approximately twenty organized mosquito-abatement districts in the State, of which six or seven are primarily concerned with the control of salt-marsh mosquitoes; the remainder are concerned primarily with malaria control.

THORNTON LEWIS,** ESQ.—Due to the fact that the Society is the oldest National Engineering Society in existence in the United States, it is fitting that its members should be interested in the very important subject of ventilation, for it is not only an engineering question, but one that vitally affects the health and comfort of all human beings. Dr. Palmer's paper is a real contribution to the literature of this subject. It is sane and well balanced. Unfortunately, only a few of the men who have attempted to discuss this subject in recent years, have had such a broad and comprehensive view of ventilation.

As a matter of fact there is a controversy raging at present between what might be called the open-window theorists and the mechanical ventilation

* Cons. San. and Hydr. Engr., Berkeley, Calif.
† Received by the Secretary, September 30, 1927.
‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1639.
§ *Am. Journal of Public Health*, Vol. 2, No. 6, pp. 452–455.
‖ *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 778.
¶ Chapter 584, Statutes of 1915.
** Pres., York Heating and Ventilating Corporation, Philadelphia, Pa.

advocates. It is not really for engineering bodies to decide this question. The Medical Profession should determine what is proper for man's health so far as air conditions are concerned. Then, the duty of the engineer is to provide these conditions. Technically speaking, the engineer should not take part in the determination of what is necessary for public health and the medical man should not concern himself with the method by which these conditions are obtained by the engineer. Unfortunately, neither of these professions has entirely done its duty in this respect.

The ventilating engineers, however, have established a research laboratory at Pittsburgh, Pa., which is operated in co-operation with the U. S. Bureau of Mines for the determination of basic facts relating to ventilation and heating. This work has been proceeding since 1919 and some very valuable results have been obtained, but the end is not yet in sight.

In this work the engineers have been ably assisted by the U. S. Public Health Service, which has made the physiological determinations, and one of the results of this very fine co-operative work has been the comfort chart. This chart shows what temperature and percentage of relative humidity gives the greatest degree of comfort. This has been termed "effective temperature". Scientists and those interested in heating and ventilating from commercial standpoints have been awaiting this chart for years. This is only the beginning, however, and with further co-operation from the Medical Profession still more light can be thrown on the troublesome question of ventilation.

Dr. Palmer mentions* the common standard of 30 cu. ft. of air per min., per person, which has been written into most of the State laws governing the ventilation of school buildings. He states that this provision is unnecessarily severe, and should be amended. Perhaps he is right. Let it not be forgotten, however, that it has taken years to secure any standard of ventilation in school buildings of the various States and before efforts are made to destroy the standards that have been set up, let the correct standard first be determined.

The proponents of open-window ventilation also state that the 30-cu. ft. standard is too large, but they do not indicate the correct amount. They fail to recognize that with the method they offer, there will be times when no ventilation will be secured, for the wind does not always blow. If there is a school building with rooms on the four sides, the wind can blow on only one or two sides, at most, at the same time. The open-window ventilation is not positive ventilation.

One prominent engineer, and an authority on the subject of ventilation, has compared open-window ventilation to the old sailing ships, which were governed purely by the whim of the wind, and mechanical ventilation to the steam-driven modern palace of the sea. Certainly, there is some analogy in this comparison.

No engineer in the country to-day, who is familiar with the question of ventilation, would oppose a change in existing laws to lower the amount of air per person introduced into schools or other buildings where human beings are

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1642.

assembled, provided the change was made to some standard that had a scientific basis.

Most ventilating engineers feel very much as the Civil Engineering Profession would feel if a physician came along and said that the factors of safety used for steel structures were wasteful and that, instead of 10, some other factor of safety, lower than this, was all that was needed. If he did not accompany this statement with any definite scientific information as to what the lower factor should be, the engineer would feel that it was better to adhere to the higher one which was known to be safe, and which certainly could not produce harm, than to allow some law to be passed that would reduce it below that where the safety of the public was assured. That is the position of most ventilating engineers to-day. They welcome any scientific information on this subject, but they will oppose the tearing down of the progressive standards already built up until those standards can be replaced with new ones based on facts and not on theories.

The speaker is forced to disagree with Dr. Palmer when he states:* "Engineers have yet to learn how to counteract the heat production of a hall full of people so that temperatures at the close will be nearer 70 than 80 degrees." The absorption of this heat production has already been solved, not only in theory but in practice, and is being used in a great many theatres and other buildings to-day. The reason it is not more generally used is that such a system is not only costly to install, but also expensive to operate. The theatre owner, however, who is largely interested in the box-office receipts, has found that in summer it will pay to use this costly system to give comfort. The further use of this system is limited only by its commercial value and not by the ability of the ventilating or air-conditioning engineer to produce these results. In the manufacture of many products, such as capsules, photographic films, artificial silk, etc., it has been found that air conditioning, even if expensive, is profitable and, therefore, it is being used. When the general public considers its own health and comfort equal in value to a manufactured article, it will be able to secure the proper air-conditioning equipment to produce the desired result.

The speaker does not mean to suggest by this latter statement that he disagrees with Dr. Palmer in his forecast of the progress to be made in ventilation; for certainly there will come tremendous improvements in this art as well as in others. However, a more intelligent public opinion is needed on the subject of ventilation and public health and a great deal more interest on the part of medical men than in the past. It is very encouraging to see the Society considering a paper of this character. It is also encouraging to observe several co-operative efforts being made by members of the medical fraternity, public health associations, and the ventilating engineers. It is only through such movements that an early solution of this very complex question can be reached.

In the last 100 years there have been radical changes. Formerly, people lived out-of-doors most of their lives, but now they have become an indoor people. Even transportation systems, such as subways and closed automobiles,

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1647.

do not allow them to be outdoors as much as they need to be. It is, therefore, necessary, as Dr. Palmer points out, to furnish fresh air for indoor conditions. The ventilating engineer needs the assistance of the physician and the chemist to tell him what quality there is in the outdoor air that makes it "fresh". He can undoubtedly provide "fresh air" if he knows what it is.

SAMUEL R. LEWIS,* ESQ. (by letter).†—The writer agrees in general with Dr. Palmer, except in his statement‡ that:

"The present air-flow standard is undoubtedly excessive. This may be decreased by one-half in such places as school rooms without discomfort or harm to the occupants".

It is hardly fair or in keeping with the temperate statements of the remainder of the paper, to imply that these sentences express the consensus of present-day opinion in regard to ventilation.

In many cases, the present air-flow standard may be excessive. The air volume required is believed now to be governed by requirements of temperature and odor control, rather than by chemical or organic substances in the air.

There is evidence to support the belief, now held by many students of the subject, that an increase in health and comfort in school rooms may be gained by a supply of air per pupil per minute considerably less than the usual 30 cu. ft.

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† Received by the Secretary, September 30, 1927.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1643.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

NEW THEORY FOR THE CENTRIFUGAL PUMP

Discussion*

By Messrs. J. W. Macmeeken, Thomson Kingsford, Sanford A. Moss, and P. L. Reed.

J. W. Macmeeken,† Esq.—The theory expressed by the author is interesting in view of the fact that other recent research and experimental work has proved the reliability of accepted theories of the centrifugal pump. This novel explanation of the performance of these pumps is possibly applicable to machines designed to operate in a particular way, but it could not be applied to many centrifugal pumps of very recent design giving higher efficiencies than had been obtained a few years ago and which show under test and analysis, a greater generation of head for peripheral speed than can be explained by the author's theory. Accurate measurements of output and input energy and measurements by laboratory methods, of each of the mechanical losses, including disk friction, show that in a pump which is designed throughout for very high hydraulic efficiency, the head obtained can only be explained by the full effect of Equation (*B*),‡ no element being omitted. Equation (*C*),‡ on the other hand, is not strictly a practical equation because it assumes suction conditions that seldom exist. The author states that the fundamental laws of centrifugal pumps seem never to have been properly explained and that many misconceptions exist, even among those supposedly familiar with the hydraulics of centrifugal pumps.§ He further refers to fallacies and proclaims, referring to Equations (*B*) and (*C*), that "they do not, and cannot, furnish a rational basis for the analysis and design of centrifugal pumps".‖ There is very strong evidence that he is wrong in discarding mathematical facts and replacing them by rational reasoning.

* This discussion (of the paper by A. F. Sherzer, Assoc. M. Am. Soc. C. E., published in October, 1927, *Proceedings*, and presented at the meeting of October 5, 1927), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1779.

§ *Loc. cit.*, pp. 1776–1777.

‖ *Loc. cit.*, p. 1779.

The increasing commercial value of pump efficiency, when obtained by designs that are economical for manufacture, resulted in a thorough study of the problem by the speaker and his associates, in America as well as with consultants in Europe. The theory was studied in a variety of ways and Equation (B) has been recognized as correct. To prove the theory experimentally, special pumps were designed and built, in order to deliver as nearly as possible the theoretical head, $\frac{V_2^2}{g}$, reduced only by the unavoidable slight rotational head of the pump suction. Some of these pumps gave a head of $\frac{0.85\ V_2^2}{g}$, or $\frac{1.70\ V_2^2}{2\ g}$. One pump, built in Europe, gave a total head of $\frac{0.96\ V_2^2}{g}$ at shut-off and practically the same at designed conditions. As 96% efficiency is the highest that can be expected, if friction and shock losses are reduced to a minimum, this pump proved that the total head, $\frac{V_2^2}{g}$, was being generated according to theory. However, these pumps were costly to manufacture and were not suitable for the American competitive field. They also showed some instability due to their susceptibility to air in the water.

Equation (B) having been proved experimentally by pumps of special design, a large number of different types and sizes were planned to include features more suitable for economical manufacture. Equation (B) was used for head calculation and every hydraulic consideration, known to result in high efficiency was introduced into the design. The pumps were built with unfinished interior water-ways; and, when tested, they not only delivered the capacity and head for which they were designed, but gave the same shape of characteristic to within 1%, or 2%, as the designer had expected. The results in these cases pointed to Equation (B), when used with knowledge of the mechanical and hydraulic losses in the pump, as giving the accuracy obtained in designing electric motors. The effect of a change in suction design could also be computed by Equation (B). These pumps were of the volute design of an improved type and the volutes were of such a form that some remaining kinetic energy is discharged into the discharge cone of the pump where it is partly converted into pressure. In all cases, the head obtained could only be explained by the full effect of Equation (B), nothing being omitted.

In the pumps referred to by the author, which have a concentric case, it is possible that this exit velocity component is not discharged but retained unobstructed in the pump-casing and, therefore, no power is used in maintaining it. This would explain satisfactorily the difference between the author's theory and Equation (B) and between efficient volute pumps and efficient pumps of the author's design. This could be expressed in the following equation:

$$H = \frac{1}{g} (C_2\ V_2 \cos \alpha_2 - C_1\ V_1 \cos \alpha_1) - f \frac{V_3^2}{2\ g}$$

when V_3 is the velocity in the concentric case and f is a factor equal to, or less than, unity and possibly zero, depending on the design of the exit orifice, $\frac{V_3^2}{2\ g}$ being subtracted because it is not allowed to leave the pump, as in a

volute type. This equation is used in the design of multi-stage pumps with concentric cases and permits the application of accepted and proved theory to both volute pumps if f is zero, and to pumps of the author's design, if f approaches unity.

The author's design will always require a greater impeller diameter than that required in the volute pump of good design. Therefore pumps of large size involving thousands of horse-power and high head may reach 35% approximately more weight and cost than is necessary, and higher disk stresses in the impeller. It is noticeable that the most efficient volute centrifugal pumps of certain types generate a head greater than $\frac{V_2^2}{2\,g}$ throughout the entire range of capacity.

The author fails to be convincing in certain respects. His illustration of the hammer thrower is not applicable. In a centrifugal pump there are an infinite number of balls, all without restraining wires and all acting on one another. The final energy is the integral of the infinitesimals between limits of entrance and exit. The same error is repeated in the illustration entitled "New Conceptions of Pump Action".* He mentions the case of a circular disk with radial grooves in which a smooth ball travels as the disk rotates. This is correct as regards the dynamics of a particle or a single mass. Were this illustration applicable to a pump with radial vanes the head would be $\frac{V_2^2}{2\,g}$ at shut-off and $\frac{V_2^2}{g}$ when flow occurs. In the centrifugal pump there is a disk full of such balls, and the total head is the sum or integral of their combined energy. Integrating between the limits of r_1 and r_2 and expressing the velocities as functions of angular rotation and radius, Equation (B) is again the result, which shows the same head, $\frac{(V_2^2)}{g}$, at shut-off and all capacities for radial vanes. The illustrations in Fig. 2* apply very nearly to a lawn sprinkler, but not to a centrifugal pump. It has been proved experimentally that a volute or guide-vane, restraining the water from flying out of the impeller, does not result in appreciable losses. Pumps so constructed show very high hydraulic efficiencies and generate high heads, much higher than $\frac{V_2^2}{2\,g}$.

There is an apparent discrepancy† in referring to Fig. 4 (b).‡ The increment of power is equal to the capacity multiplied by the head. This means that the efficiency at small capacities is 100%, which is impossible if the hydraulic efficiency is considered. Certain losses are proportional to $\frac{V_2^2}{2\,g}(1 - k)$, in which, k is the ratio of the actual capacity considered to the designed capacity. These losses, therefore, will be great at very low capacities.

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1780.
† *Loc. cit.*, p. 1784.
‡ *Loc. cit.*, p. 1783

Referring to the objection to guide-vanes: These are not necessarily used to reduce the velocity, C_2. They have a useful function like that of a volute in controlling the output from the impeller. They prevent the water from flying out from the impeller and can do so with little reduction in efficiency; and they assist in obtaining high peripheral speeds with low impeller velocities in cases where a volute presents structural difficulties.

Referring to the author's explanation of the performance of a volute pump (Fig. 8),* an entirely different explanation could be given. Equation (B) may be transposed into the form:

$$H = \frac{V_2^2}{2\,g} + \frac{C_2^2}{2\,g} - \frac{W_2^2}{2\,g} - \frac{V_1^2}{2\,g} - \frac{C_1^2}{2\,g} + \frac{W_1^2}{2\,g}$$

In the event that,

$$\frac{C_2^2}{2\,g} - \frac{C_1^2}{2\,g} = \frac{W_2^2}{2\,g} - \frac{W_1^2}{2\,g}$$

then,

$$H = \frac{V_2^2 - V_1^2}{2\,g}$$

which would explain the performance. The method used by the author† is incomplete in that it takes no account of the ratio of Dimension A to Dimension C. According to his method only the product of Dimensions A and C is effective. It is known that a change in the ratio of Dimension A to Dimension C will affect the characteristic curve very greatly.

The circular case pump, referred to by the author‡ has an efficiency of 79 per cent. The commercial efficiency of present-day pumps designed for this condition is 77%, not 60%, as stated by the author. Its efficiency could be raised to 79% by exact workmanship in the foundry and extra good mechanical condition. A good commercial pump of this type does not show falling off of the efficiency with increase of speed as shown in Fig. 12.§ When there is an obstruction in the volute, the efficiency has been found to fall with increased speed, as shown in Fig. 12.

The pump in Fig. 14‖ shows an efficiency of 78 per cent. A commercial pump, of the same specific speed type and about the same size, is rated at 80 per cent.

In the case of multi-stage pumps the circular case has been used by many manufacturers for about sixteen years at least. Impellers of the same design have been found to give very much greater efficiencies in multi-stage volutes. This indicates again that the volute is superior.

Much misunderstanding has arisen regarding calculations for centrifugal pumps because old designs were inefficient compared with later machines in which, 88.5% and even higher efficiencies have been obtained on very accurate tests.

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1791.

† *Loc. cit.*, pp. 1790-1792.

‡ *Loc. cit.*, pp. 1794-1795.

§ *Loc. cit.*, p. 1795.

‖ *Loc. cit.*, p. 1796.

Undefined losses led designers into empirical design constants that had no dynamic basis for their value. It is only when high efficiencies are obtained, that the losses can be analyzed with accuracy and the fundamental equation used in its theoretical form.

A centrifugal pump, developing less head than that expressed by Equation (*B*), is a machine that is leaving uncompleted work and represents over-expenditure of material for work done. This is of great importance in very large pumps and might even warrant the restriction of impeller diameter, by contract specifications.

Large pumps, like large hydraulic turbines, are generally designed for as high a rotative speed as possible, with only a small margin of safety below the speed at which cavitation would occur. It has been proved that the design of the volute is important in avoiding cavitation. At very high speeds and heads a circular case with an orifice outlet is known to be noisy in large machines.

Equation (*B*) has a very wide field of application and is used by designers in hydraulic turbines, hydraulic meters, fans, and blowers. In each application designers have used their ingenuity to obtain the best results. The speaker has found it not only a useful and reliable means of designing centrifugal pumps, but also a means of analyzing results and obtaining indications of possible improvements.

THOMSON KINGSFORD,* ESQ. (by letter).†—The author deserves great credit for the way he has handled his subject, and for the results he has obtained. The writer has had occasion to check several pumps against the principles given in the paper, and finds that the tests agree substantially with the results given in the author's experiments. Unquestionably the efficiency found in pumps designed according to these principles as recorded in the paper is high, considering the pump size and other conditions of operation.

The author's Fig. 16‡ is particularly interesting as a comparison between two similar pumps, one of the old type with volute casing, and the other with the circular casing, as indicating the increased head obtained by the circular design. He does not state the differences in efficiency, but presumably this is in favor of the circular casing; at least it should be. Of course, the increased head is due to the specially designed impeller and larger casing. A comparison between the corresponding efficiencies would be of interest.

SANFORD A. MOSS,§ ESQ. (by letter).‖—It is true, of course, that there are many misunderstandings in connection with the mathematics of centrifugal pumps. However, the fundamental equations, as usually given, are correct. If they are used by any one who really understands what they mean, they lead to correct results. The author mentions a new analysis, but he gives no detailed equations. It is quite probable that if these detailed equations were

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† Received by the Secretary, October 11, 1927.

‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1798.

§ Thomson Research Laboratory, General Electric Co., West Lynn, Mass.

‖ Received by the Secretary, October 14, 1927.

included, they could be shown to be exactly the same as those in current texts, properly interpreted, so that there would really be nothing new.

One item must be clearly borne in mind, that is, that a set of equations, or a set of computations, written for flow through a centrifugal pump, applies to a condition with a single load or quantity of flow. The vanes at the impeller inlet and the diffuser inlet (or volute angles if there are no diffuser vanes), must suit this one condition. This condition is a single point on the pump characteristic curve, presumably at the point of maximum efficiency. Other points on the curve—including conditions when the pump is shut off or running without flow, and when discharge is wide open—are those that appear when the various vane angles are incorrect. Hence, analysis for these conditions cannot be made on the same basis as that where maximum efficiency is expected.

Another item that must be borne in mind is that the set of equations for the occurrences in the pump apply primarily to the pump impeller extending from the impeller vane inlet to the impeller vane exit. At the impeller vane exit is a circular entrance to a region beyond the vanes, which may be called the diffuser or volute. At the entrance to this region and at the load or flow for which computations are made, there is definite pressure due to centrifugal force acting in the impeller, and a definite absolute velocity. Each of these effects represents energy put into the fluid by the impeller. The existence of a portion of fluid under the pressure found at the impeller exit means that a certain amount of energy was exerted to get the fluid into this region. The absolute velocity of the fluid similarly represents a given amount of kinetic energy. The fluid at the point in question certainly has the two additive amounts of energy in question. If the author means to point out that it is a fallacy to so consider the matter, his conclusion is incorrect.

The author gives frequent mention of a new design of pump which he has built and which shows signs of promising results. However, no details are given showing the nature of this design. The results obtained are so interesting that a desire is created for a description of details. Readers of a technical paper always like to have all the facts that are necessary to enable them to make use of the information given by the author.

P. L. Reed,* M. Am. Soc. C. E. (by letter).†—The author has probably noted that centrifugal pumps under test do not produce nearly as great a head as that computed by rational formulas based on the generally accepted theory, which is, as he states, that for very small rates of flow the water, as it leaves the periphery of the impeller, has a velocity nearly equal to that of the tips of the vanes and, hence, a velocity head of $\frac{U\,a^2}{2\,g}$; and that it has, in addition, a pressure head of the same amount, or a total head of $\frac{U\,a^2}{g}$. With considerable rates of flow the total theoretical head is modified by the

* Capt., C. E. C., U. S. N., Bureau of Yards and Docks, U. S. Navy Dept., Washington, D. C.

† Received by the Secretary, October 21, 1927.

shape of the vanes, but there is still a marked difference between the theoretical head and the actual head.

He meets this condition by boldly asserting that the fundamental theory, which is supported by Rankine, Church, and many other authorities, is all wrong and that for small discharges the water, as it leaves the impeller, has a velocity head about equal to $\frac{U\,a^2}{2\,g}$, or a pressure head of the same amount, but not both; and that the true theoretical maximum total head is, therefore, only one-half that computed according to the accepted theory.

The accepted theory is too well grounded in the laws of mechanics to be thus overthrown and the author's position will not be sustained. With radial vanes, the prevailing theory considers that the water just inside the tips of the impeller vanes has a velocity approximately equal to that of the periphery of this impeller, which can be diverted into a volute or guide-vanes and converted in large part into pressure. If conversion were perfect, the pressure thus converted from velocity would be $\frac{V^2}{2\,g}$, or $\frac{U\,a^2}{2\,g}$.

The author does not doubt that there is pressure inside the impeller. This pressure will not (disregarding friction) be lost during the subsequent diversion of the water into the volute or guide-vanes and the partial conversion of its velocity head into pressure head. There is undoubtedly both a velocity head and a pressure head at all points in the impeller. Both these heads increase as the distance from the shaft increases; caused to be sure, simultaneously and by the same medium, the rotation of the impeller.

A number of the author's original mathematical derivations and conclusions will be found to be unsound. One is the "Derivation by Principle of Angular Momentum",* where the force, which is undoubtedly distributed over the length of the vanes of the impeller, is "conveniently regarded as concentrated" at their tips. This error exactly offsets or compensates for ignoring all centrifugal effects in this derivation, and the correct formula results.

The difference in the action between the round ball in the disk and the water in the impeller is that in the former case the centrifugal force is opposed only by the inertia of the ball, which receives a high radial velocity, while in the latter case the water is under restraint and is given a low radial velocity against resistance.

The author is misled by considering that the present pump theory is purely kinetic. The velocity head, represented by free radial velocity of the water produced by an impeller without a chamber, is not absent, but is represented by a pressure head in the encased impeller of a pump.

He presents his new theory of three orifices.† As a rational theory it does not appeal. It implies that the head required to produce the velocity at the entrance of the impeller vanes is 100% lost in passing through the impeller; that the head required to produce the velocity at discharge from

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1778.

† *Loc. cit.*, p. 1789.

impeller is 100% lost in passing around the volute; and that the velocity head at the throat is 100% lost in passing into the discharge pipe; or, at least, that the sum of these three hypothetical losses plus the loss at no discharge is the total loss of head in the pump. The only rational feature of this theory, which the writer can appreciate, is that the head losses due to discharge increase with the square of the quantity discharge. As an empiric formula it seems to have much merit when applied to the three pumps tested by the author. Whether it would be equally satisfactory if applied to pumps of different proportions would have to be developed by further experiments, which is the defect of all empiric rules.

Having pointed out what are believed to be fundamental defects in the author's theory, the fact remains that the actual head of a pump, operating with no discharge, is approximately $\dfrac{U\,a^2}{2\,g}$, and that this head is never much greater when there is a discharge (unless the pump has forward curved vanes), although the conventional application of the established theory would give a head nearly twice as great (for radial vanes). It will not do to say that the difference between the head, thus computed, and the actual head is represented by losses in friction, eddies, etc., as this would imply that the losses were nearly 50% of the total; whereas the actual measured efficiency of the pump in raising water often exceeds 70 per cent. With these facts at hand, attention is invited, not to the established theory of vortices, of which the centrifugal pump is a modified example, but to the conventional application of this theory to centrifugal pumps.

It has been noted that the actual head developed is considerably less than the rationally computed head and that this difference cannot all be explained as loss since it is not reflected in the efficiency of the pump. It must then be explained by some incorrect assumption in the application of the fundamental theory to the actual operation of the pump. The conventional application assumes that the water leaves the impeller in a direction, with respect to the impeller, parallel to the tips of the vanes. Now, if the water in the outer part of the impeller lags behind, if it spills to some extent over the following impeller vane, the effect would be similar to bending back the impeller tips, thereby reducing the theoretical total head of the pump, and reducing, if not removing, the discrepancy between conventionally computed and actual heads. This seems plausible for other reasons. The water is being constantly pressed and accelerated tangentially by the vanes as it passes through the impeller; and near the periphery it may escape this pressure in part by lagging behind and passing through the periphery at a sharper angle than the backward curved tip of the impeller vane and at a lower absolute velocity than is conventionally assumed. This would be particularly the case with a small discharge, which would accord with test results. This effect would be increased by any cohesion or viscosity that the liquid may possess, since the tangential velocity of the liquid just outside the impeller is no doubt considerably less than it is inside.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE HYDRAULIC DESIGN OF FLUME AND SIPHON TRANSITIONS

Discussion*

By H. B. Muckleston, M. Am. Soc. C. E.

H. B. Muckleston,† M. Am. Soc. C. E. (by letter).‡—The flow of water in an open channel of variable section is such a complicated phenomenon that a really exact analysis by mathematical methods is more an ideal than a possibility. By making various assumptions of more than doubtful validity, it is quite possible to arrive at a result, seemingly correct to any number of decimal places, but which is not in fact any more accurate than the assumptions on which it is founded. Present knowledge of hydraulic phenomena is entirely empirical and the resulting formulas bristle with coefficients intended to account for the many factors that are either unknown in their action or else not susceptible of measurement. Such important factors as roughness of perimeter and shape of cross-section cannot be defined in any physical unit of measurement; their influence can only be expressed by some numerical factor, chosen in the light of individual or collective experience. In view of this condition, a paper such as this, based as it is on the observation of an organization with a long and diverse experience, cannot help but be a valuable addition to engineering knowledge.

Surface Curve.—If the flow through a transition is certain to be always well above the critical depth; or, in case of an accelerating transition, whether it is or not, it is the writer's experience that time and labor are saved, with no sacrifice beyond the appearance of accuracy, if the energy gradient be assumed outright as a beginning. The hydraulic dimensions of the flume or siphon and of the canal are supposed to be known; and, therefore, the elevation of the

* This discussion (of the paper by Julian Hinds, M. Am. Soc. C. E., published in October, 1927, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Vancouver, B. C., Canada.

‡ Received by the Secretary, October 24, 1927.

energy gradient above the bottom of the channel is known at two points and the assumption of the total loss fixes, at once, the elevation of the bottom at the end of the transition.

The total loss of energy in the transition is usually divided, as in the paper, into friction losses, so-called, and conversion losses. In both cases they are really the losses due to the formation of eddies and whorls in the water, and the real cause of most of these is to be found in the irregularities of the surface, whether minute as in the friction loss or relatively large as in the conversion loss. It may be convenient in analysis to separate these two forms of the same thing, but the writer does not believe that they can be separated in experimental determinations. For a reasonably uniform cross-section, the friction loss in a given channel can be predicted with some approach to certainty; but nobody knows much about the law governing friction loss in channels of varying cross-section, even when the variation follows some simple mathematical rule; and still less when the variation follows no ascertainable rule whatever.

Very little is known about conversion losses even in circular channels. Whatever the law may be, experience with closed channels seems to show that it is not the same for converging and diverging channels; but whether these laws, if they were known, would hold for open channels is doubtful. Probably they would not, as the conditions are radically different. For this reason, complicated analysis of friction and conversion losses does not seem a justifiable labor. The losses can only be calculated by a cut-and-try process anyway, and the final result depends so much on assumptions that it is quite as logical to assume the losses in the first place if one has a little experience to guide him. When something like finality has been reached, there may be more logic in detailed calculation, but it is doubtful.

With the energy gradient fixed, the water surface profile follows and the remainder of the process is a matter of adjustment by cut-and-try to obtain smooth curves for the sides and bottom. The first selection of a surface profile may result in unsuitable shapes and dimensions, in which case a new profile must be assumed and the process repeated. If a raised floor is necessary, as mentioned later, this will fix the elevation of one point on the floor and more or less determine the profile; or if a hydraulic notch is required, it will fix the cross-section at one point and influence those throughout the transition.

Velocity Head.—The apparent velocity head is somewhat less than the real velocity head. From a few gaugings, in which the distribution of velocity over the cross-section had been observed, the writer has estimated that the difference may amount to from 1 to 6%, with a mean value of about 4 per cent. The law of the increase is not known, but as the shapes of the section and the mean velocity at various points is the last thing known, the exact law is not of importance. The writer suggests that a mean increase of about 4% would be close enough. It is unimportant in the lower velocities but more so in the higher. For a velocity of 8 ft. per sec. it is 0.04 ft., which is more than the total friction loss in the whole transition.

Accelerating Transitions.—A well-designed transition should prevent the occurrence of harmful velocities in the canal above at all stages in supply. If a transition is designed with only the conditions at full supply in mind, the surface drop is the difference in the velocity heads plus whatever is allowed for loss, and this fixes the elevation of the bottom at the end of the transition. If, now, conditions at less than full supply, say, at half depth, are investigated, and a back-water curve is calculated beginning at water level in the flume for that stage, it may happen that the water level thus found for the entrance to the transition is far below the theoretical water level for that stage in the canal. The result is that acceleration begins a long way up stream and scouring velocity may be attained before the water enters the transition. To avoid this condition, it is necessary to constrict the entrance of the water in such a way that acceleration does not begin until the transition is reached for all stages of supply. To do this absolutely for all stages would involve expensive form work for the sides and bottom, which would not be justified by the accuracy of the calculations. A notch can be designed with straight sides and walls, that will, in theory, be correct for any two stages and, in practice, sufficiently correct for all stages. If the end velocity in the transition is above the critical velocity for all stages of supply, it is not very material which stages are chosen as the bases for designing the notch; the result is a true, or very nearly true, hydraulic notch, and the bottom will be at the same level as the bottom of the canal.

If the end velocity is below the critical for all stages, the two stages should be full supply and about half depth (not half supply). The resulting notch will have its bottom raised above the level of the canal bottom. Direct calculation is not practicable, and a cut-and-try process brings results more quickly.

If the end velocity is above the critical at full supply stage and below it at lower stages, the two stages chosen should be: For the higher, that stage at which the end velocity passes the critical; and, for the lower, about half that depth. There may be some acceleration in the canal at full supply, but probably not sufficient to be dangerous. If the discharge at which the end velocity passes the critical is less than about half supply, the transition may be designed as a hydraulic notch. In this case, a jump will develop in the flume at the lower discharges, but that will be of no great importance.

When the end velocity is below the critical at full supply and above the critical at lower stages (a condition common in siphon transitions), the upper design stage should be chosen as full supply and the lower at that stage where the end velocity passes through the critical. In extreme cases the notch may be quite narrow and its bottom may be depressed below the level of the canal. When this occurs, uncomfortable acceleration may be expected at very low stages and some rip-rap may be needed as a cure.

Decelerating Transitions.—If the flow in the high velocity channel is at less than critical velocity, the process of design is much the same as for accelerating transitions. The length should be much greater than is necessary for accelerating transitions. In fact, experience indicates that converging

channels may be quite obtuse without great loss in efficiency; but if even moderately obtuse angles are used for diffusing channels, the water shows a tendency to stick to the center for a long distance down stream.

Decelerating transitions should be investigated for conditions at partial supply. It is quite possible for a transition that acts with satisfaction at full supply to give strong evidence of scour at lower stages. The reason is that all the deceleration does not take place in the designed length, and there is still an excess of velocity remaining when the water leaves the transition.

If the flow in the high velocity channel is below the critical depth, it is almost impossible to design a transition that will surely effect a smooth deceleration through the critical region. A jump is almost certain to occur; and as it involves a loss of head it is better to design for the loss. Otherwise, the result may be an overflowing flume.

Siphon Transitions.—At siphon inlets the design follows the same general principles as for flumes. Outlets may be designed on the same principles as flume outlets if the velocity is not greater than the critical velocity for the same quantity in the canal. If it is greater, the deceleration should be accomplished under pressure until the critical region is comfortably passed.

The length of the diffusing section may be shortened without sacrifice of efficiency if the flow through the diffuser is forced to take place along spiral lines. As shown by Mr. F. zur Nedden,* the efficiency of a polished cone with an angle of 8° 20′ was raised from 88.3 to 98.9% by so doing, and it is probable that a similar result would follow in a cone of much larger angle. If this is true, the same efficiency might be expected from a relatively obtuse cone with spiral flow as from an acute cone with straight flow.

* "Induced Currents of Fluids," *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 844.

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PAPERS AND DISCUSSIONS

A GRAPHIC METHOD FOR DETERMINING THE STRESSES IN CIRCULAR ARCHES UNDER NORMAL LOADS BY THE CAIN FORMULAS

Discussion*

By B. F. Jakobsen, M. Am. Soc. C. E.

B. F. Jakobsen,† M. Am. Soc. C. E. (by letter).‡—The real value of this paper lies in the information it affords about the relation between $x = \frac{t}{r}$, the central angle, $2\,\phi_1$, and the stresses. For stress calculations the curves and tables prepared by William Cain, M. Am. Soc. C. E.,§ were quite sufficient, but the curves prepared by the author give an excellent picture of what happens when either $\frac{t}{r}$, or $2\,\phi_1$, or both, is varied, and, therefore, they should assist the designer by enabling him to see more quickly what may be accomplished in a specific case by varying the design in a certain direction. The writer has found it necessary, when designing arch dams, to plot for a limited region such curves as the author submits, but has never made a systematic investigation, such as that presented by Mr. Fowler.

In analyzing the stresses in the Kerckhoff Dam at Elevation 900, the author's solution involves a tension of 165 lb. per sq. in.,‖ which should exist between the concrete and the rock abutment. The existence of a tension of this magnitude between concrete and rock, however, is highly improbable and, at any rate, something on which a careful designer would not rely. The

* This discussion (of the paper by Frederick Hall Fowler, M. Am. Soc. C. E., published in October, 1927, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, October 14, 1927.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233; also, Vol. 90 (June, 1927), p. 522.

‖ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1917.

usefulness of the Cain formulas is due to the fact that in such cases a fairly accurate estimate of the maximum stress can be obtained by applying the theory of the secondary arch.*

Referring to Fig. 14, *F D B N* is the original arch, which has tension at, and around, Point *D* and possibly also at, and around, Point *N*. Assuming that the arch cannot withstand any tension, the concrete will move away from the rock abutment, beginning at Point *D* and continuing to a point, *E′*, where the stress is zero. The load is now carried on the secondary arch, *F E′ B M*, and this secondary arch is determined by the fact that the stress at Point *E′* is zero, when the water is assumed to act on the arc, *F E′*. The actual load is on the arc, *F D*, but water may also be assumed to act on *D E′* and these two loads combined equal the water load on Arc *F E′*.

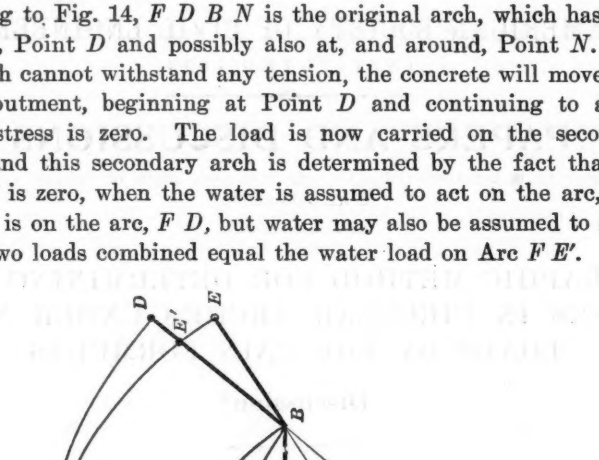

FIG. 14.

From the author's curves in Figs. 5,† 12(*a*)‡ and 12(*b*),§ it may be seen that for any value of $x = \dfrac{t}{r}$, there is a definite value of the central angle that gives zero stress at the abutment of the up-stream face. If the central angle remains the same, but $\dfrac{t}{r}$ is increased, tension will occur at the abutment of the up-stream face and *vice versa*. Consequently, a curve may be plotted from these limiting values of $x = \dfrac{t}{r}$ and $2\,\phi_1$. Such a curve has been plotted

* "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 510, where reference is made to the work of L. J. Mensch, M. Am. Soc. C. E., Prof. Cain, and Prof. Résal.

† *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1902.

‡ *Loc. cit.*, p. 1912.

§ *Loc. cit.*, p. 1913.

in Fig. 15 by utilizing the author's curves. As an illustration, for $x_0 = \frac{t_0}{r_0} = 0.3$, Fig. 15 gives $2\,\phi_0 = 142°$ for the limiting central angle, and this angle was obtained from the author's Fig. 12(a) for $\frac{t}{r} = 0.3$ and zero stress.

In general, it is necessary to try several secondary arches before the right one is found. By determining a constant, B, as a function of $\frac{t}{r}$ and $2\ \phi_1$ and also as a function of the limiting values, x_0 and $2\,\phi_0$, and plotting B in Fig. 15, the secondary arch can be determined directly. The following considerations lead to the curve, B, in Fig. 15.

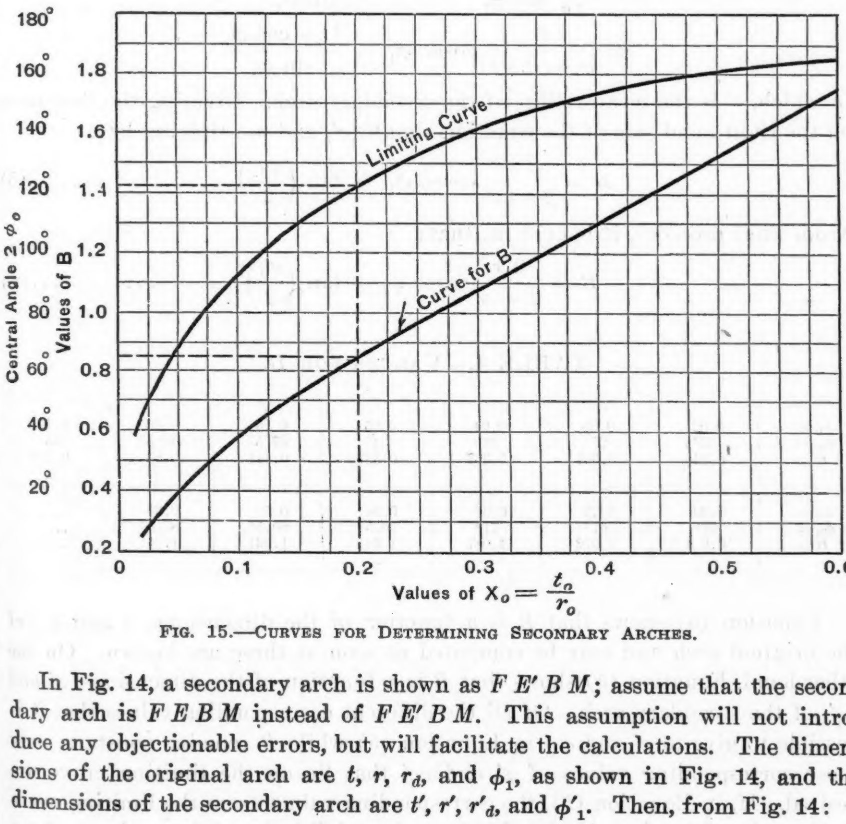

FIG. 15.—CURVES FOR DETERMINING SECONDARY ARCHES.

In Fig. 14, a secondary arch is shown as $F\,E'\,B\,M$; assume that the secondary arch is $F\,E\,B\,M$ instead of $F\,E'\,B\,M$. This assumption will not introduce any objectionable errors, but will facilitate the calculations. The dimensions of the original arch are t, r, r_d, and ϕ_1, as shown in Fig. 14, and the dimensions of the secondary arch are t', r', r'_d, and ϕ'_1. Then, from Fig. 14:

$$A\,B = r_d \sin \phi_1 = r'_d \sin \phi'_1$$

or,

$$r'_d = r_d \frac{\sin \phi_1}{\sin \phi'_1} \quad \text{.........................(1)}$$

Also, from Fig. 14:

$$F\,A = t + r_d\,(1 - \cos \phi_1) = A \quad \text{.................(2)}$$

Let,

$$B = \frac{A}{r_d \sin \phi_1} \dots\dots\dots\dots\dots\dots\dots\dots (3)$$

The value of A can be determined from Equation (2) as soon as the dimensions of the original arch are known and, likewise, the value of B may be determined from Equation (3).

A and B, however, may also be expressed in terms of the dimensions of the secondary arch. From Fig. 14:

$$F A = A = t_d' + r_d'(1 - \cos \phi_1') \dots\dots\dots\dots\dots\dots (4)$$

From Equations (3) and (1):

$$B = \frac{A}{r_d' \sin \phi_1'} = \frac{t' + r_d' (1 - \cos \phi_1')}{r_d' \sin \phi_1'}$$

$$= \frac{t'}{r' - 0.5\, t'} \operatorname{cosec} \phi_1' + \frac{1 - \cos \phi_1'}{\sin \phi_1'}$$

in which, r' is the mean radius of the secondary arch. Dividing the first term on the right-hand side of the equation sign by r', and multiplying by 2:

$$B = \frac{2\, x'}{2 - x'} \operatorname{cosec} \phi_1' + \tan \left(\frac{\phi_1}{2}\right) \dots\dots\dots\dots\dots\dots (5)$$

From what precedes, it is evident, that:

$$B = \frac{2\, x}{2 - x} \operatorname{cosec} \phi_1 + \tan \left(\frac{\phi_1}{2}\right) \dots\dots\dots\dots\dots\dots (6)$$

TABLE 4.—VALUES FOR B.

$x_0 =$	0.02	0.03	0.04	0.05	0.06	0.10	0.15
$\phi_0 =$	22°	27°	31°	34°	37.5°	46.5°	55°
$B =$	0.246	0.307	0.357	0.397	0.441	0.60	0.719
$x_0 =$	0.20	0.25	0.30	0.40	0.50	0.60	.....
$\phi_0 =$	61°	67°	71°	76.5°	80.5°	82.5°	.....
$B =$	0.8 4	0.972	1.087	1.301	1.521	1.741	.....

Equation (6) shows that B is a function of the dimensions, x and ϕ_1, of the original arch and may be computed as soon as these are known. On the other hand, Equation (5) shows that B is a function of the dimensions, x' and ϕ_1', of the secondary arch. Of all the different corresponding values that it is possible to give to x' and ϕ_1' in Equation (5) while B remains constant, only those corresponding values of x' and ϕ_1' that lie on the limiting curve are desired. If, in Equation (5), the corresponding values from the limiting curve are introduced in place of x' and ϕ_1', a value of B is found for each set of the values, x_0, ϕ_0. In Table 4, x_0 was assumed, ϕ_0 was found from the limiting curve in Fig. 15, and the value of B was then computed from Equation (5). The value, $\frac{t}{r} = 0.1$, belongs both to Fig. 12 (a) and to Fig. 5, and a mean value has been used in order to obtain a smooth curve.

The stresses in an arch, expressed in terms of the water pressure on the extrados, p_e, are completely determined when $x = \frac{t}{r}$ and ϕ_1 are given. Any set of values of x and ϕ determine a definite point in Fig. 15. If the point is located above the limiting curve, as, for example, the arch, $x = 0.1$ and $2\,\phi_1 = 120°$, there is compression throughout the arch. If the point lies on the limiting curve, there is compression throughout the arch, except at the point, *E*, of Fig. 14 where there is zero stress. If, finally, the point lies below the limiting curve there is tension at the extrados of the abutment and possibly, also, at the intrados at the crown. In this case the theory of the secondary arch must be utilized in order to determine the maximum stress. This is accomplished by computing the value of B from Equation (6) and then, from Fig. 15, determining the corresponding values of x_0 and $2\,\phi_0$. Then, x_0 and $2\,\phi_0$ is on the secondary arch for the given primary arch, and the maximum stress, which occurs at the intrados of the abutment, may be taken from the author's curves, bearing in mind that the stresses found in the curves are for a 10-ft. head.

For example: Suppose the original arch gives $B = 0.845$; then, from Fig. 15, using "Curve for B", it is found that $x_0 = 0.2$ and the limiting angle, which corresponds to $x_0 = 0.2$, is $2\,\phi_0 = 122$ degrees. This is the secondary arch. If the conditions at the abutments are not as shown in Fig. 14, some modifications may be necessary, but the writer has found the two curves shown to be of considerable value in actual design, even for very thick arches, although for these the yielding of the abutments must be taken into account.*

The question proposed by the author,† "What are the stresses in the Kerckhoff Dam?" may now be solved with the understanding that the maximum stress found is most likely somewhat in excess of the actually existing maximum stress, but will be a good guide in design and quite reliable except perhaps for very short thick arches. At Elevation 900, the dimensions are $x = \frac{t}{r} = 0.201$ and $2\,\phi_1 = 82°\ 40'$; this gives $B = 0.7154$ from Equation (6), and from "Curve for B", in Fig. 15, is found $x_0 = 0.145$ and $2\,\phi_0 = 108$ degrees. These are the dimensions of the secondary arch (r_d' can be determined from Equation (1)) and the maximum stress may now be read from Fig. 13(*a*)‡; this gives 66 lb. per sq. in. for a 10-ft. head and 627 lb. per sq. in. for a head of 95 ft. The stresses at the crown may be found by referring to the author's diagrams. Generally, however, the stress at the intrados of the abutment is the maximum stress occurring and if there is not tension at the extrados of the abutment, there is no tension in the arch; that is, when the stresses are determined by the Cain formulas and the yielding of foundation is neglected.

* See discussion by Dr. Fredrik Vogt, *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 554.

† *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1916.

‡ *Loc. cit.*, p. 1914.

The author, as he states,* took the dimensions of the Kerckhoff Dam from the writer's discussion of Professor Cain's paper,† in which the water surface was considered as standing 10 ft. above the Taintor gates. When designing the Kerckhoff Dam, the writer took as the maximum possible head, that which might come on the dam if all the gates were closed and an extreme flood of about 100 000 sec-ft. was flowing in the river. The actual maximum operating head is only 85 ft. on the section at Elevation 900, the top of the Taintor gates being at Elevation 985;‡ therefore, the maximum stress is found to be:

$$\frac{627 \times 8.5}{9.5} = 561 \text{ lb. per sq. in.}$$

When the writer, in May, 1919, took charge as Designing Engineer for the Kerckhoff Development of the San Joaquin Light and Power Corporation, Fresno, Calif., he found a gravity arch dam design, that would have cost $1 000 000 to build. The saving of the arch dam as designed by the writer amounted to approximately $400 000, or 40% of the total, and the stress calculations show that the factor of safety of the arch dam is much greater than for most gravity dams.

The curves plotted by the author also give information as to how the maximum stress varies when the central angle is held constant, while the ratio, $\frac{t}{r}$, is varied. For example, for $2\,\phi_1 = 140°$, $\frac{t}{r} = 0.02$, 0.05, 0.10, and 0.20, the maximum stress being, respectively, 237, 107, 66, and 44 lb. per sq. in. for a 10-ft. head.

In other words, when $\frac{t}{r}$ is increased 10 times, from 0.02 to 0.20, the stress is decreased only 5.4 times, or, the arch, $\frac{t}{r} = 0.02$, is nearly twice as efficient as the arch, $\frac{t}{r} = 0.20$, when the central angle is 140 degrees. This gives an indication of the great importance of utilizing a high strength concrete produced under competent inspection. The writer's experience while in charge of the construction of the Pacoima and Big Santa Anita Dams,§ and the experience of other engineers, have shown that when the water-cement ratio is kept under close control, it is possible to produce a high grade and uniform concrete under actual construction conditions. The fact that the thinner arch is so much more efficient than the thick arch, is bound to have a considerable influence on concrete production.

The curves presented by Mr. Fowler are a welcome addition to the literature of stresses in arches and should be helpful in the design and study of arched dams.

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1916.

† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 258.

‡ *Loc. cit.*, Vol. LXXXIV (1921), p. 107.

§ *Loc. cit.*, Vol. 90 (June, 1927), p. 585.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE DELAWARE RIVER FROM PHILADELPHIA TO THE SEA

Discussion*

By Messrs. Charles M. Spofford, Henry J. Sherman, and Earl I. Brown.

Charles M. Spofford,† M. Am. Soc. C. E.—This paper impresses the speaker as a most enlightening contribution to engineering literature relating to the regulation of rivers. The feature of greatest interest to the speaker is the theory on the "constancy of mean depth" originally suggested by the late Henry Mitchell, M. Am. Soc. C. E., and adopted by various members of the U. S. Engineer Corps who have studied the river.‡

The author§ states that no two investigators agree as to the theoretical formulas for the contraction required to produce a certain depth of channel; that there still seem to be insurmountable obstacles in determining where dikes should be located; and that, in practice, it is necessary to adopt the "cut and try" process and to permit considerable time to elapse before sufficient knowledge of results can be obtained to warrant further progress. These conditions make pertinent the inquiry as to why small scale experiments on models of the river have not been made to determine the validity of the theoretical conclusions before spending large sums in construction. The money required for experimental tests would seem to be so insignificant compared with the cost of unsatisfactory construction that such means of investigation should be ignored only for the most potent reasons.

Experimental work of this character has been conducted for many years in German universities. Apparently, it has been well worth while, judging

* This discussion (of the paper by F. C. Boggs, M. Am. Soc. C. E., presneted at the meeting of the Waterways Division, Philadelphia, Pa., October 8, 1926, and published in October, 1927, *Proceedings*), is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Hayward Prof. of Civ. Eng., Mass. Inst. Tech., Cambridge; Cons. Engr. (Fay, Spofford & Thorndike), Boston, Mass.

‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1968.

§ *Loc. cit.*, p. 1981.

by the fact that in recent years several additional important laboratories, equipped to make such experiments, have been constructed and many experimental investigations made on German rivers.

The speaker inspected two of these European laboratories in May, 1926, the State Laboratory in Vienna, Austria, and the laboratory at the Technische Hochschule at Karlsruhe, Germany, in both of which he found much activity. At Karlsruhe, experimental work was being conducted under Professor Rehbock on the River Maas, in The Netherlands, and on the Quadälquiver, in Spain, and the speaker understood that experimental investigations had also been made in this laboratory on certain South American rivers. In recent years, a large and expensively equipped laboratory has been constructed in Stockholm, Sweden, and complete plans have been prepared for an extensive laboratory for this purpose at the Technische Hochschule at Zurich. Evidently, other countries than Germany are alive to the wisdom of such experimental investigations. Complete accounts of the experiments that have been performed in the various European laboratories have been published.* In this country, no such laboratory as yet exists, but the Massachusetts Institute of Technology is seriously considering the construction of one.

The figures given for the cost of dredging† seem exceptionally low and it would be interesting to know whether these figures include all charges, such as overhead, interest on investment, salaries of engineers and officers connected with the work, depreciation, insurance, and other factors. The question arises, if dredging by Government plant is done along the lower reaches of the river, why is it not also economical in Philadelphia Harbor where dredging is now done by contract? Possibly this is due to the fact that, while constant dredging is required in the lower reaches of the river, the dredging in Philadelphia Harbor is only intermittent.

In the paper, the construction of dikes, both as a means for controlling the channel and also for holding the dredged material within given areas, is mentioned.‡ It would be of added value if information was presented as to the method of constructing these dikes; particularly as to whether wooden bulkheads have been used. It would seem, because of the comparatively soft material of the bottom at some locations, that the construction of dikes would be decidedly expensive.

Considerable attention is given to the commercial advantages resulting from the improvement of the Delaware River, thus establishing Philadelphia as a first-class port.§ The Port of Philadelphia is one of a notable group of river and canal ports which include such important ports as London, Hamburg, Antwerp, Montreal, New Orleans, Baltimore, Manchester, and Houston. These ports, all of which are some distance from the sea, and many of which have required large expenditures for improvements, are among the busiest in the world. Apparently, inland ports are in favor with shippers, but just how far the Government should go in the development of river ports in a

* "Die Wasserbaulaboratorien Europas."

† *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1977.

‡ *Loc. cit.*, p. 1978.

§ *Loc. cit.*, p. 1983.

country where natural harbors of deep draft, requiring but moderate expenditures for maintenance and improvements, already exist, is difficult to determine. Why should the Government spend large sums in improving the Port of Philadelphia when considerable freight passing through this port could equally well be shipped from Boston, for example, which is at least one day nearer Europe and which possesses an admirable natural harbor and excellent shipping facilities?

It is true that, in time of war, interior ports are safer against destruction by enemy battleships, although they are not much better protected against airplanes. On the other hand, they are more easily obstructed by sunken vessels and are sometimes greatly hampered by ice. Even a port as far south as Baltimore was seriously hampered by ice in 1918. If private capital is to be used in developing a port, no one can justly protest. By the expenditure of large sums of private capital a self-sustaining port has been developed at Manchester, England, all ships entering it proceeding directly past the Liverpool docks. Even in this case, one naturally questions whether, if the same sum had been expended in Liverpool and upon the railways connecting Liverpool and Manchester, it would not have been for the economic advantage of the country.

HENRY J. SHERMAN,* M. AM. SOC. C. E.—A cross-current passing over a shoal area and thence across a channel deposits its burden of material, carried in suspension, into the latter. No doubt in the Delaware River, as elsewhere, this is one of the principal causes of shoaling. Movement of a soft bottom in shallow waters, caused by passing vessels in narrow channels, is a constant source of shoaling. It is fortunate that there is now available complete and exhaustive studies of the tides and currents from a high source like the U. S. Coast and Geodetic Survey, on which to base future operations.

In Table 1,† showing monthly duration of rise and fall of tides, it is observed that the greatest variation is 0.22 hour for rise, or 4.50%, while it is only 0.12 hour for fall, or 1.60%, the monthly average being decidedly constant, despite the great variation in single cycles noted by the author.

A further study of Fig. 2‡, showing annual variations in high and low water, reveals the maximum difference of 0.4 ft. for low water and 1.10 ft. for high water, the latter being nearly three times the former.

In Table 2,§ the differences in the tidal ranges on the two sides of Delaware Bay are so large as to be worthy of mention, being no less than 1.76 ft. at Maurice River; a large percentage of the total range.

In Table 3,‖ it is shown that high water in New Castle, Del., has been raised 0.38 ft. as compared with only 0.05 ft. and 0.07 ft. at Philadelphia, Pa., and Marcus Hook, Pa., respectively, and that the low-water plane at

* Cons. Engr., New Jersey Board of Commerce and Nav.; Cons. Engr. (Sherman & Sleeper), Camden, N. J.

† *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1964.

‡ *Loc. cit.*, p. 1965.

§ *Loc. cit.*, p. 1967.

‖ *Loc. cit.*, p. 1968.

Philadelphia and Marcus Hook has been raised exactly the same amount, namely, 0.53 ft. Opinions will probably differ as to the causes of raising the low-water plane, a number believing that the contraction of the river is the answer. The figures given for cost of dredging seem very low in comparison with contract work, even after allowing for contractor's profit, and marine and liability insurance, items which, it is understood, are not included in these figures.

The fact that the 26 and 35-ft. channels have been self-sustaining near Cherry Island, without reduction in the width of the river,* as theoretical studies had indicated would be necessary, is a vindication of a conservative policy in dike construction. The summary† of the obstacles preventing dike construction is excellent. Specific mention might also be made of the high cost of installation and maintenance of such structures.

The author speaks of the strong opposition of shore interests cut off by a dike, and particular reference was made to citizens of New Jersey stopping the completion of Fisher's Point Dike. In this connection it may be said, there is now a demand for the removal of the outer portions of that dike and the re-opening of the channel back of Petty Island. In September, 1926, the War Department held a hearing to obtain views on proposed harbor lines at this point. This channel would accommodate prospective commerce in the upper part of the City of Camden. When the dike was proposed many years ago, the riparian lands were of little value; the navigation interests cut off were small; not much interest was shown in the project; and no public body was charged with the duty of studying the effect of the jetty on State, or local interests. To-day, however, with the great increase in the value of riparian lands, and State and municipal bodies watchful of all water-front changes, such strong opposition would develop against the building of any dike that the project would probably be abandoned. Opposition is so strong to another dike that is partly completed that a provision was actually inserted in the River and Harbor Bill prohibiting the use of any of the funds for the continuance of that dike.

Under Obstacle (*c*),‡ the author states: "The navigability of a channel depends on the governing depth of that channel". This is now 32.7 ft., so near the project depth, that further dike construction might appropriately be undertaken if this were the only obstacle.

Under Obstacles (*d*) and (*e*),‡ it is observed that where longitudinal dikes have been constructed, the channel in every case is maintained in straight ranges and, in some cases, is a considerable distance from the dikes.

The author makes timely reference to control of some piers by the municipality.§ Manufacturing and small shippers who cannot afford to locate on an expensive water-front should, nevertheless, have those facilities and be assured access to a pier at all times at reasonable rates.

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* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1973.

† *Loc. cit.*, p. 1978.

‡ *Loc. cit.*, p. 1982.

§ *Loc. cit.*, p. 1983.

Under "Economics"* the interest rate has been taken at 4.50%, which appears somewhat high. A rate of 4% or even 3.5% as an average of Government borrowing over a long term of years might be fairer.

The author has ingeniously worked out a cost of 21.4 cents per ton to the United States for the increased tonnage due to the deepening of the channel beyond the actual depth of 18 ft.† While, undoubtedly, this figure is high compared with Northern rivers, it probably would compare favorably with the alluvial streams of the South.

While the paper is directed specifically to the part of the Delaware River from Philadelphia to the sea, reference might be made to the proposed ship canal across New Jersey from Bordentown to Raritan Bay, as the estimated tonnage from this source will utilize a large portion of the river under discussion. The State of New Jersey, believing the United States is almost ready to undertake construction, has made an appropriation to begin purchase of the right of way.

The author has compiled the important data and engineering opinions of the best minds in the U. S. Engineer Corps and Staff bearing on the problem of regulating the Delaware River. Supplementing this are his own observations and conclusions on certain features of the work. His presentation of the arguments for and against dike construction is eminently fair and unbiased. Out of the controversy which has been waged on this question, a middle course has been adopted utilizing each method to some extent. Careful thought and study have been given to the expenditure of the funds committed to the War Department, and the results achieved justify the confidence reposed in those charged with the work.

In his plain and comprehensive review of the work accomplished and analysis of the problems involved, he has made a splendid contribution to the literature of river and harbor improvement.

Earl I. Brown,‡ M. Am. Soc. C. E.—The author is to be congratulated upon the very complete justification he has given of the policy of placing more dependence upon dredging to secure immediate and positive increases in depth of channel than upon regulation by training walls and dikes. Not that walls and dikes should be neglected, but they should be constructed only as a supplement to dredging, after an exhaustive study of commercial requirements as to channel depths, and of the susceptibility of the river to improvement by regulation. Some rivers lend themselves to it more readily than others, the character of the bottom, the shape of the bed, and the amplitude of the tide being factors.

Many engineers, not themselves engaged in active practice of river improvement, but reading in current engineering literature of the extensive and expensive regulating works constructed in the tidal rivers and harbors of the Old World, and comparing them with the scarcity of such works in practice

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1983.

† *Loc. cit.*, p. 1987.

‡ Col., Corps of Engrs., U. S. A.; Engr., Eighth Corps Area, Fort Sam Houston, Tex.

in the United States and the greater reliance here placed on dredging for these improvements, are tempted occasionally to protest that river engineers in this country are not utilizing regulating works as they should.

The tide and its resultant phenomena are the means whereby the regulating work in a tidal stream achieves its ends. Compare the usual ranges of 15 to 20 ft. in European seaports with the average range of 4 to 5 ft. in ports of the United States, and it will be appreciated that this country could not expect to accomplish the same result with the same structure. A study of tidal conditions and manifestations are not a matter of such vital importance to America.

It is not to be understood, however, that it is the speaker's purpose to minimize the importance of tidal studies in connection with the improvement of rivers. He believes, on the contrary, that it could and should be carried out to a greater extent than it usually is. If the results of a full and complete tidal study are available, the engineer can prepare his project and execute his work with a greater degree of confidence in the ensuance of benefit to navigation.

That part of the paper relating to studies made upon physics and hydraulics of the river* does not indicate that a great amount of attention has been paid to these very important questions. However, it must be conceded that any improvement project must be based on a study of them if the work is to be accomplished in the most successful manner. Failure to plan the improvement so as to take full advantage of the knowledge derived from those studies is a step in the dark, which may be injurious.

Previous investigators only went so far as to endeavor to find a law of variation in the width and cross-section of the river, based on an assumption that the mean depth is the same in all sections. These investigators proceeded on empirical rather than theoretical lines, and found equations of either the parabolic or the hyperbolic type. It is now generally conceded by river engineers that the law of variation of width of a tidal river is best expressed by a logarithmic equation. Recent investigations show that there is a definite relation between the variation of the energy of the tidal wave and the width of a tidal river. If the bottom be horizontal, as reported by Mitchell and assumed by the other investigators, the equation is of the simple exponential form as a function of the distance from the mouth of the river. If the bottom has a horizontal slope, the law of variation is more complicated, and is in the form of a compound exponential equation.

In most estuaries and tidal rivers, the amplitude, the speed of the wave, that is, its rate of propagation, and the velocity of the currents diminish more or less rapidly going up stream, because of the expenditure of the energy of the wave as a pure loss in overcoming the friction that results from the insufficiency of the mean depths and from the excess width of the bed, as well as from obstacles which it meets. By lessening these obstacles, it is possible not only to provide for the utilization of that energy for the conservation of the amplitude, the speed, and the velocity of the wave, but also to measure them

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1968.

to some extent by suitable means. All these factors have a bearing on the cost and probable degree of success of any project, and they cannot be neglected without paying the penalty in increased improvement and maintenance costs.

The Artificial Island mentioned by the author* is an example. The curves showing the range of tides in the Delaware River as given in Fig. 4† emphasize a marked falling off of range of tide in the period 1890-1900 on that part of the river between Woodland Beach and near the upper tidal limit. Colonel Boggs indicates this falling off, but gives no explanation of it. However, by reference to the same source quoted by the author,‡ a statement is found that this change is to be attributed to the improvements effected in the river under the projects in force.

This explanation is not satisfactory, and cannot be accepted. The dredging of shoal areas beneath the plane of low water would have no effect on the height of tides, and the volume of material excavated from above that plane would not be sufficient, quantitatively, to account for the decreased volume of tidal flow above Woodland Beach. Some other explanation must be found.

The drop in range occurs in the reach between Woodland Beach and Delaware City, Del. Some of the most extensive contraction works on the Delaware River have been constructed in this reach. These include Artificial Island and the dikes connecting it with the mainland, and a dike running down stream from Reedy Island.

In his study of the law of variation of widths, H. F. Flynn, M. Am. Soc. C. E., gives the data shown in Table 7.

TABLE 7.—CROSS-SECTION AREAS, DELAWARE RIVER.

Mile.	Mean area of section in 1881-82, in square feet.	Measured area in 1916, in square feet.	Theoretical area, as computed by formula, in square feet.	Decrease in area, 1881 to 1916, in square feet.	Present deficiency in area, in square feet.
31	248 700	241 100	256 000	7 600	14 900
32	263 200	248 000	267 000	15 200	19 000
33	301 400	236 000	279 200	65 400	43 200
34	324 500	249 000	291 500	75 500	42 500
35	321 700	241 000	304 700	80 700	63 700
36	317 300	261 000	318 900	53 300	57 900
37	331 600	323 800	334 100	7 800	10 300

It will be noted that the greatest contraction of area is at Mile 35, near the lower end of Artificial Island, where, expressed in percentages, the original channel was about 5% in excess of theoretical requirements. It has been contracted to 75% of its original width, and has only 80% of its theoretically required width.

A contraction of this magnitude in the bed of a tidal river, the bottom of which is mobile, would cause scouring to the extent that the cross-section might be thereby restored approximately to its original size, and no material

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1974.

† *Loc. cit.*, p. 1972.

‡ *Special Publication No. 123*, U. S. Coast and Geodetic Survey, 1926, pp. 43 and 47.

damage would be done in the long run to the tidal propagation. However, the bed of the Delaware River at this point cannot be classed as mobile. It is a very hard stiff mud which, when dug out by a bucket dredge, retains its shape and consistency. The river, therefore, has not been able to erode its bed and restore the magnitude of its cross-section. Consequently, the loss of energy due to this contraction is still felt in a decreased tidal range above the point of contraction. The injurious effects of this great contraction could be largely obviated, and the cross-section restored to a great degree by the removal of the two dikes which connect the island with the mainland.

It may be contended that this reduction in tidal range manifests itself, as it usually does, in a raising of the plane of mean low water in the upper section of the river, thus rendering increased depths available at that stage for navigation. This is a pertinent reply, and the resultant benefit must be balanced against those that would follow the increased tidal range, with its greater current velocity, and hence greater facility for self-maintenance.

It has been predicted by some engineers that a permanent improvement of the Delaware River by dredging could not be successfully accomplished, as there was a tendency to constancy of mean depth throughout the estuary. The author quotes the late Henry Mitchell, M. Am. Soc. C. E., to that effect,* and also refers† to the recommendation of H. C. Ripley, M. Am. Soc. C. E., that for dredging there be substituted a change of straight reaches into channels of suitable curvature by means of training walls to secure depths suitable to navigation needs.

The very satisfactory results that have been obtained by dredging are sufficient refutation of adverse predictions on that score, and Colonel Boggs indicates the many serious practical objections to a sinuous channel. From a theoretical point of view the increase in constant mean depth thereby obtained, will result in increased amplitude, speed, and velocity of the tidal wave, thus enabling the river to maintain the increased depth, when once obtained, by reason of the improved tidal conditions. There is, however, a limit beyond which tidal conditions can not be improved, and artificial maintenance will have to suffice. It is not known at the present time where that limit is.

As between straight and sinuous tidal channels, theoretical as well as practical considerations lead to a preference for the former. There is less friction and, consequently, less loss of energy in a straight than in a sinuous channel, and it is the preservation of the tidal energy to the farthest limit up stream that is to be sought. The principles enunciated by M. Fargue relating to curvature and depth of channel, apply to streams flowing in one direction, but not at all satisfactorily to tidal streams. One of the greatest bends on the Delaware River, that just below New Castle, Del., is not accompanied by any perceptible deepening, and the channel lies on the convex point, not in the concave bend.

The author has touched upon shoaling as a result of lack of coincidence in certain places of the thread of the flood and ebb currents. It would have

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1969.

† *Loc. cit.*, p. 1971.

been very interesting and instructive if he had given a statement or tabulation showing the points where there is such a lack of coincidence, and had indicated the comparative amount and cost of maintenance dredging per mile at such places as compared with the same amounts in channels more suitably conditioned. It is the general impression that the larger part of the annual maintenance dredging is now done on a few bad places, perhaps not to exceed six. If that is a fact, it indicates the necessity of such a tidal study as herein referred to with a view to finding means of eliminating those conditions.

Summing up, the speaker's views are, that the general policy under which the improvement of the Delaware River is being done, is correct, and is based on sound principles, although some errors of judgment have been made in the locations of channels and of regulating works (including disposal grounds when in the bed of the river). A detailed study of the conditions of tidal propagation in the river might have prevented some of these errors, and might now enable them to be corrected, with a strong probability that maintenance costs could thereby be lessened, and future errors largely avoided.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

TRAFFIC CONTROL BY ELECTRIC SIGNAL LIGHTS

Discussion*

By Messrs. L. J. Carmalt and E. P. Goodrich.

L. J. Carmalt,† M. Am. Soc. C. E.—Like all papers on this subject written by those who have had practical experience in handling traffic matters, that of Mr. Eldridge is a valuable contribution. The magnitude of the subject under discussion, however, is such that conclusions which are not of general application should not be adopted as final even if it is assumed (which Mr. Eldridge does not explicitly state) that they are successfully applied in Washington.

The writer has to disagree with Mr. Eldridge's conclusions as to the amount of responsibility to be placed on the pedestrian.‡ In his discussion regarding the use of the amber light, Mr. Eldridge recommends a cycle in which the amber light is omitted after the red and retained after the green. In this the writer heartily agrees, but differs decidedly as to the length of the amber period. A 5-sec. interval will most certainly not give ample protection to the pedestrian, except under certain rather limited conditions.

The ordinary pedestrian walks at about a speed of 3 miles per hour and this is reduced in a crowd. There is also always a considerable percentage of those, who from age or other reasons, find it difficult to maintain even this rate. Four miles per hour is a hurrying pace and cannot be kept up unless there is freedom to move straight ahead without having to consider foot obstructions, such as rough surface, car tracks, curbs, etc., or moving obstructions, such as other pedestrians or vehicles. At street crossings where traffic is busy enough to require signals he has all these to contend with and, in addition, he is expected to keep an eye on the signals. Therefore, a pace of 3

* This discussion (of the paper by M. O. Eldridge, Assoc. M. Am. Soc. C. E., presented at the meeting of the Highway Division, New York, N. Y., January 20, 1927, and published in October, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New Haven, Conn.

‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1992.

miles per hour is as much as can be expected of pedestrian traffic and the time for crossing a roadway should be based on this speed, covering about 4½ ft. per sec.

Contrast this with what is allowed for the motor vehicle. Mr. Eldridge uses a speed of 20 miles per hour to illustrate a proper time interval for cars to clear a crossing. This is equivalent to about 30 ft. per sec. and a 5-sec. period allows a car to cross even the widest roadway in Washington—Mr. Eldridge, in discussion, gave 75 to 90 ft. as that of Pennsylvania Avenue—with a second or two to spare for being within 50 or 60 ft. of the crossing when the amber light is flashed. Table 1 will show more clearly the time needed by the two classes for different street widths.

TABLE 1.—Time Studies.

Distance between curbs, in feet.	Number of Seconds Needed for Crossing.	
	At 3 miles per hour (4½ ft. per sec.).	At 20 miles per hour (30 ft. per sec.).
20	4.5	0.7
30	6.6	1.0
40	8.8	1.3
50	11.1	1.7
60	13.3	2.0
70	15.5	2.3
80	17.7	2.6
90	20.0	3.0

In other words, Table 1 shows that the vehicle has the advantage in speed in about the ratio of 7 to 1.

In the discussion following the reading of Mr. Eldridge's paper, the author made the statement "that it is just as important for the pedestrian to know the time limit as it is for the vehicle [driver]". The writer disagrees with this because, granting equal responsibility for pedestrian and driver, the time interval (that is, the number of seconds) is not what controls the driver; certainly not outside of Washington, and it is doubtful if even there one driver in a hundred can give it correctly. He does know that there is a cycle of green, amber, red, amber; and that, when proceeding on the green, he must clear any crossing before the following amber goes off. He also knows that with permissible speeds he can cross any street in ample time if he is within 50 to 60 ft. of the crossing at the moment when the amber light flashes. All he has to keep in mind about the lights is whether they are for or against him as he approaches, and watch for the change.

The pedestrian, on the other hand, knows that if the amber light flashes soon after he has stepped from one curb he cannot reach the opposite one, except on very narrow streets, before the red light will be set against him and that the vehicles on the cross street will have the right to start across his path. On narrow roadways, less than 30 ft., he may have the time, without undue hurrying; for if such streets carry a heavy enough traffic to justify signal lights, the speed of vehicles is necessarily slowed down to considerably less

than 20 miles per hour. Therefore, when a pedestrian starts to cross a street with the signal in his favor, he has to make some calculation or guess as to how much of the green period remains and make his own determination as to whether it is sufficient, together with the amber period, to cross with safety. The burden of this mental process, which is not required of the driver, not only is placed on those who would suffer most in case of a collision with a vehicle, but is required of a mass of people who cannot be expected to have the necessary reactions to changes in the colors of the lights unless given a training in the meaning and use of traffic signals somewhat equivalent to that imposed on the drivers.

To make the pedestrian equally responsible, therefore, with the driver for clearing the crossings, requires giving him equal rights, and this is not done if he is given only the same length of time for this purpose as is given the driver. Knowledge of the number of seconds in the periods is beside the mark, whether required of the driver or of the pedestrian. That is a function of those who control the traffic and should be determined by study of actual conditions, not only at each particular crossing, but also in relation to those of other crossings along the routes in the vicinity. In this connection, Mr. Eldridge's description of the system in use on 16th Street, in Washington, is very interesting and well worth studying.

In the report of the Second National Conference on Street and Highway Safety held in Washington in March, 1926, the following are among the recommendations for "Traffic Laws and Regulations":

"In cities pedestrians should be instructed, urged and required to keep within the boundaries of designated safety zones and crossing places and, when there is congestion, to cross only with the traffic. Motorists should be required to accord pedestrians safe and dignified use of such safety zones and crossing places. Pedestrians as well as motor vehicle operators should be required to obey the traffic rules and regulations and should be punished by adequate fines for failure to do so."

The writer maintains that the Washington regulations, as interpreted by Mr. Eldridge, do not conform to the spirit of these recommendations.

As a surer method of regulating traffic for the safety of pedestrians the writer suggests: (*a*) Vary the time interval for the amber light to that required for the average pedestrian to cross the vehicular lanes; and, (*b*) where this would require an excessive length of time, establish isles, or zones, of safety in the center of the roadway, permitting the pedestrian to cross the wider streets during two separate periods.

This would give due consideration to the rights, convenience, and safety of each class of traffic. As stated, the actual time periods are a function of local conditions. They should be founded, however, on the principles mentioned of a fair adjustment of relative responsibilities and should be based on the human, as well as the mechanical, ability to "carry out" definitely worded regulations.

E. P. Goodrich,* M. Am. Soc. C. E.—One difficulty in traffic control is the cost of installing the system. Mr. Eldrige has mentioned the need, at the

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present time, of running a separate cable from each central control point to each intersection. A patent is now pending for a scheme that will require only one pair of wires from the central station to any area. The remainder of the plan, with reference to the change of lights, to the interposition of a fire signal system, to an all-night flashing—amber, for example—can be arranged over the two wires from the central station.

From some experience in arranging traffic control systems and studying traffic problems in various cities, the thought has occurred to the speaker that, in a great many cities, too many traffic signals are being installed. Doubtless all have had the experience that they can get along the street faster without a policeman than with one at an intersection.

In one city, in which a great deal of work has been done and where traffic signals have lately been installed, the speaker has found that he could progress without danger of accident and without any trouble of any other kind very much faster before the traffic signals were installed than afterward.

In other words, the fad to-day is traffic signaling. It would seem that there should be some criterion as to when a signal should be installed at an intersection. The following is suggested. When the benefit to the community (to the traveling public primarily, of course), is greater than the loss involved through the operation of the signal system, then install it. After it is installed, if any signal is going to decrease traffic (for example, stop a series of vehicles whether or not there is any cross traffic), then that particular signal should not be operated at that time of day.

In other words, do not install until there will be some benefit in reducing accidents and in reducing the total delay caused by congestion; and do not operate unless that same criterion can be followed. It is believed that will very largely reduce the cost of installation and the cost of operation; and, at the same time, it will benefit traffic, because it will not stop it when unnecessary to do so and will expedite it correspondingly.

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PAPERS AND DISCUSSIONS

BOUNDARY SURVEYS

Discussion*

By Messrs. Frank M. Johnson, Henry J. Sherman, G. M. Bowers, Robert L' H. Tate, and E. A. Van Deusen.

Frank M. Johnson,† M. Am. Soc. C. E. (by letter).‡—Land boundaries, both political and private, always have played a part in the material development of nations, and by reason of their function in defining and protecting property have contributed in a broad sense to the ultimate learning and philosophy of mankind. The author presents this truth against a historical background, and points to some of the benefits that might have accrued to posterity if scientific consideration in larger measure had been accorded in the beginning to the political sub-division of the United States and the earlier cadastral surveys of its public lands.

Colonial Boundaries.—If more extended engineering study had been made in selecting the limits of the major political units in this country a more scientific co-ordination of administrative, commercial, and industrial effort ultimately would have resulted. Development history, however, is a chronicle of failure and success, regulated by the economic stress of the times. The boundaries of the States and Territories were essentially controlled by, and often wholly subject to, influences quite unrelated to engineering considerations. The form of territorial acquisition, the conflict of indefinite original boundary descriptions, and the uncompromising requirements of National expansion, were factors that largely determined the jurisdictional limits in the then new and rapidly growing country.

One need but recall the interminable controversies over questions of territorial dominion created by the vague and incongruous boundary descriptions of the royal grants of territory in America, to appreciate the weight

* This discussion (of the paper by C. T. Johnston, M. Am. Soc. C. E., presented at the meeting of the Surveying and Mapping Division, New York, N. Y., on January 20, 1927, and published in October, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† U. S. Superv. of Surveys, General Land Office, Interior Dept., Denver, Colo.

‡ Received by the Secretary, January 20, 1927.

of contending forces in the settlement of such matters. It is doubtful whether, in the original Colonies and States, engineering consideration could have played any appreciable part in the fixation of their limits. Probably not, as decision in each case rested largely on the legal construction of the original charters, supported by such collateral evidence as existing sovereignty and common interest. The Mason and Dixon Line, for example, although surveyed on the ground by two eminent English engineers and astronomers, was first determined as to location and type by a British Court of law.

International Boundaries.—It is a long forward stride in the matter of boundary definition from the day when a conqueror with an army at the gates of a city dictated without restraint the limits of the enemy territory he intended to acquire, to the modern jurisprudence of political treaty or covenant arrived at by duly authorized commissioners and formally ratified by the interested countries, subsequently to become the supreme law of the land. It may be said that for 100 years or more, in the settlement of international affairs involving boundary lines to which the United States has been a party, the best scientific as well as legal and economic thought of the day has been employed. The Boundary Commissioners themselves usually have been, as a matter of course, men of repute, trained in international law and diplomacy, as the fixation of a common line between nations raises many major problems of National intercourse and human rights. The strictly engineering features of the final boundary position within the meaning of the treaty have been left largely to their technical staffs. Under the provisions of the covenant entered into by this country and Great Britain, July 22, 1892, and the supplementary convention of February 3, 1894, this country, owing to the obvious impracticability of making an acceptable demarcation of the coastal boundary between Alaska and British Columbia, made extensive preliminary surveys in the field for the purpose of securing enough information to permit intelligent consideration by the Joint Commission of the question of the permanent line between the two countries. Scientific effort was applied in this instance in its most practical form, not only in the subsequent establishment of the line upon the ground, but in the deliberations preliminary to the final decision as to location.

Western State Boundaries.—A different principle controlled the selection of the State and territorial boundaries in the western areas of the United States. The Government exercised paramount jurisdiction over the public domain, which was encumbered with no condition except that it should be held and used and disposed of for the common good of the nation. The newly acquired territory, at first that of the Louisiana Purchase and, later, the Mexican and other cessions, was organized by Congress into territories of large extent under natural boundaries. These gave way with the tide of advancing settlement to smaller units which, subsequently, as population and material development warranted, were organized into States with definite limits. Boundaries so fixed were necessarily designed to meet the exigencies of a precise and urgent economic demand at times when accurate geographical

information in essential detail concerning the Far West was not always available.

The reports and maps of the early American explorers, who themselves were scientists or were accompanied by scientific staffs, were definite only as to limited areas or routes and were inadequate in themselves as bases for any comprehensive plan of political sub-division. Moreover, development with the discovery of gold was so rapid and far-flung that even if the country had been awake to the full significance of jurisdictional boundaries, it is questionable, in view of the practical requirements of the fast-growing communities, whether any more extended program of scientific study of the subject would have been either practical or desirable.

As it was, Congress fixed the Western State boundary lines from time to time with consideration of such information as it had before it, supplemented by the representations of the applicant jurisdictions themselves. Any one familiar with the history of such legislation must know that conclusions were not easily or hastily reached. The advice of Government and other scientists was freely sought. In most instances boundaries were fixed along parallels of latitude and meridians of longitude, a course which might point to the conclusion that the influence of Jefferson's carefully devised and strongly advocated theory of rectangular States was still felt in later boundary line deliberations. Although his theory was never adopted, its corollary, the scheme of rectangular surveys of the public lands was carried into practice.

Lines were surveyed and monumented on the ground as material expansion suggested, the older original boundary surveys west of the Mississippi, with few exceptions being executed by surveyors and astronomers, in accordance with the accepted standards of scientific procedure of the time, under contract with the General Land Office. In comparatively recent years other linear boundaries, were surveyed by engineering agencies of the Government. These may be called "original" boundaries in the sense that they were fixed, in whole or in part, independently of earlier incomplete or erroneous demarcations of the same boundary. The geodetic line between California and Nevada was established by the U. S. Coast and Geodetic Survey in 1893 to 1899, and the boundary between Idaho and Montana on the Thirty-Ninth Meridian was surveyed by the U. S. Geological Survey in 1898 to 1899. In this type of re-survey there also might be included the Texas and New Mexico Boundary, fixed by commissioners authorized to act for Texas and the United States, in 1911 to 1912.

It will be conceded that, from the viewpoint of National economic and political values, a more equitable distribution of territory and natural wealth among the States than the existing plan affords, would be desirable. This truth is particularly obvious from the viewpoint of some of the States themselves, and applies in some measure perhaps to every political jurisdiction in the world. Existing boundaries, however, are not apt to change except in the event of future sub-division of some of the larger States. Like the streets of an ancient city laid out in relation to an enclosing wall which long ago disappeared, they continue to serve the original purpose in a modern environment.

From the viewpoint of scientific field procedure under the existing boundary line plan, the results quite naturally would have attained the ideal if it had been possible, throughout the past century, to have referred State lines to the first-order triangulation of the U. S. Coast and Geodetic Survey on the North American Datum; and to have employed in their establishment the refinements of technical procedure now in general use. As it is, many astronomically established State lines are defective in varying degree in position and bearing, due partly, if not entirely to station error. Moreover, a few of them are lost or obliterated in parts. All such lines, when once established and accepted by proper authority remain, in the absence of joint action by Congress and the interested States to the contrary, the boundaries of the jurisdictions to which they pertain. They may be materially in error in position, as disclosed by suits before the Courts over the latitudinal line between Virginia and Tennessee; the meridional line between Maryland and West Virginia; and, in lesser degree, the Thirty-Seventh Parallel of North Latitude between Colorado and New Mexico. Such lines, however, are not subject to change. The Supreme Court holds that,

"Governments (states) as well as individuals are bound by the practical line that has been recognized and adopted as their boundaries," and, further, that "a boundary line between Governments (states) which has been run out and marked upon the earth and afterwards recognized and acquiesced in for a long course of years is conclusive, even if it be ascertained that it varies somewhat from the correct course."

Present-Day Problem.—It is apparent, therefore, that the present-day problem in regard to State lines is one of restoration where need be, and of preservation of existing legal boundaries, rather than one of their re-adjustment to true position, however desirable such a course might be from the purely scientific standpoint. Substantial progress is being made in this field, although there is yet room for a more thorough appreciation of the importance of the subject on the part of local authorities of many of the Western States.

It is well known that sections of boundary lines crossing isolated stretches of desert and mountainous regions have become practically obliterated, and because of the apparent absence of necessity for any pressing jurisdictional action in their vicinities, have been neglected or forgotten. The discovery of some new use for the adjoining lands usually brings in its wake an endless train of litigation over property rights. A small fraction of the Court costs, would have paid for re-establishing monuments and maintaining the line. Periodical boundary line inspection and report, as practiced in some of the Eastern States, should be adopted throughout the country and rigidly enforced under engineering direction.

From the Federal viewpoint, the engineering agencies of the U. S. Geological Survey and the General Land Office, in the interests of administrative and other necessity, have retraced and re-established, in large part, the boundaries of several Western States; while smaller segments are being restored from time to time for the purpose of closing public land surveys. As indicated, in cases of controversy as to the location of common boundaries between States resulting in suits in equity, the Supreme Court of the United States

has original jurisdiction; and through its uniform practice in recent years of selecting its boundary commissioners from the ranks of recognized cadastral, geodetic, and topographic engineers, has made a notable record of scientific achievement in this field and established a precedent most gratifying to the engineering world.

HENRY J. SHERMAN,* M. AM. SOC. C. E.—As in many other fields of human progress, engineers are indebted to the Romans for the first crude map of which there is a record, showing natural and artificial features of a given area from actual measurements. Their extensive possessions were largely connected by roads, now famous; and it is generally believed they used an instrument like a plane-table for determining their alignment. However, instruments adapted to surveying purposes were in use many centuries before the Romans came into power. It is asserted that in 1600 B. C. the Chinese possessed a crude magnetic compass, but historians do not know their method of linear measurements, or whether they reproduced their "field work" on paper.

A cadastral survey for taxation was in use in Babylon in the age of Sargon of Akhad, 3800 B. C. In the British Museum are a series of circular clay tablets dating about 2300 to 2100 B. C., which contain surveys of land. One indicates in a crude manner, Lower Babylonia enclosed by a "salt-water river", Oceanus.

The ancient Egyptians in the days of Rameses II (1300 B. C.) had prepared a cadastral survey of their country, showing the rows of pillars that separated the nomes and also the boundaries of estates. It was on a map of this character that Eratosthenes (276-196 B. C.) measured the distance between Syene and Alexandria for determining the length of a degree, to which the author has referred.† Apollonius, of Rhodes, successor to Eratosthenes as Chief Librarian at Alexandria (196 B. C.), states in his "Argonautica" that the inhabitants of Colchis, descendants of Egyptian colonists, preserved tablets on which land, water, roads, and towns were shown.‡

These are a few additional historical facts, in connection with early surveys and maps, which may be of some interest, but boundary surveys in this country are of more immediate importance. A system for the severance of the primary land title into individual ownership, which is believed to be unique, is that of the surveys, or grants of land by the Proprietors of New Jersey, an organization still extant. Some early history that led up to the organization seems pertinent.

In 1664, James, Duke of York, granted the territory of New Jersey to Lord John Berkeley and Sir George Carteret, James having acquired it from Charles the Second, his brother, in the same year. In 1669, Carteret and Berkeley agreed upon certain concessions, and Philip Carteret was appointed Governor of New Jersey. He was given power, with the advice of the Council, to grant lands to all such as were entitled thereto, exercising his prerogatives for a period of 11 years.

* Cons. Engr., New Jersey Board of Commerce and Nav.; Cons. Engr. (Sherman & Sleeper), Camden, N. J.

† *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 2013.

‡ See Encyclopædia Britannica.

In 1675, Lord Berkeley sold and conveyed his half part of New Jersey to John Fenwick in trust for Edward Byllinge and his assigns. Byllinge became involved in financial difficulties and deeded his interest to William Penn, Garwin Laurie, and Nicholas Lucas, as Trustees, for the benefit of his creditors. These Trustees agreed upon a division with Carteret. The Trustees and Proprietors and John Fenwick, who set up a claim for a tenth part, took the western division, known as "West New Jersey", and Carteret, "East New Jersey". It is quite generally believed that Fenwick asserted his right to a tenth part for holding title in trust for Byllinge. What follows refers particularly to the Province of West New Jersey. The territory was divided into 100 shares, Fenwick receiving 10 shares, which were set off in the vicinity of Salem. The Trustees sold enough land to satisfy Byllinge's creditors, giving him the remainder and making him the Chief Proprietor.

Under what is known as "Concessions and Agreements of Proprietors, Freeholders and Inhabitants of West New Jersey," in 1676, commissioners were appointed *inter alia,* "to take care for setting forth and dividing all the lands of said province as be already taken up, or by themselves shall be taken up or contracted for with the natives". A plan for the "expedition" of laying out land "to be planted and settled upon" and to mark the same in the Register and upon some of the trees, etc., was also suggested. It was also provided that surveyors appointed by the Proprietors, or Commissioners, may survey all such lands as shall be granted from any of the Proprietors to the freeholders, planters, or inhabitants, and a particular or tenor thereof to certify to the Register and be recorded. Thus, it would seem that machinery was constituted for surveying, returning, and registering as early as 1676.

The Council of Proprietors was organized in 1686. It was difficult for the Proprietors to travel long distances, so they decided to meet annually and elect nine to represent them; five from Burlington and four from Gloucester. It is believed that the Proprietors generally met to pass on the early surveys, as the records show that surveys were ordered returned at the next "Cort". Directions were evidently given in these early days. The Council elected a Surveyor General. The procedure to secure a grant to a survey was as follows: The Council of Proprietors issued a warrant stating the acres of rights on the strength of a proprietary share or on a void survey when John Jones "craves" a warrant to * * * acres. Having this, an order was issued by the Surveyor General to one of his "lawful" deputies. The deputy made a survey of the outbounds and furnished a description of it, which was known as a "return", and a map to the Surveyor General who inspected the survey, proved the calculation, and corrected any errors he might find.

About 1795 to 1800, owing to gross frauds, boundary calls were eliminated, all surveys since then depending on a full and complete description of the beginning corner only. This step was taken despite the fact that boundary calls have been given precedence over distances since the beginning of survey descriptions. The Proprietors discovered that Deputy Surveyors were purposely short in the calls. One survey of 444 acres actually contained 25 000

acres. One of 650 acres was divided into 35 equal lots, a single lot containing 3 000 acres. In another survey retraced, the lines were four times as long as the calls, with all the corners marked, thus making the area sixteen times that for which the Proprietors were paid.

It is unfortunate for those who are obliged to retrace these ancient surveys that boundary calls were eliminated and dependence placed on the beginning corner only, since the prominent objects to which the beginning is referred are frequently destroyed. The Board of Proprietors is still active, meeting at least once a year and making occasional grants. There are approximately 13 000 Proprietary Surveys in West New Jersey.

The Proprietors acquired the entire State, including the lands under water and the powers of Government. In 1702, they surrendered the governmental function, but did not undertake to give up the lands under water. Until 1860 active ownership of lands under water was assumed by the Proprietors, and many grants of oyster grounds were made by them.

While the Proprietors feel that their title to lands under tide-water should be recognized, the U. S. Supreme Court has upheld the State's ownership as the successor to the Crown. It was not until 1851, however, when what is known as the "Wharf Act" was passed, that the State undertook to assume any general control. This Act, as the title implies, "authorized owners of land upon tide waters to build wharves in front of the same". Legislative grants giving individual owners or companies certain areas were made as early as 1802.

In 1869 by an act of the Legislature, the custody of the riparian lands was placed in the hands of Commissioners appointed by the Governor. The Wharf Act as applied to the Hudson River was repealed the same year, but it was not until 1891 that the repeal was made operative over the State at large. As funds were provided, the Riparian Commissioners had surveys made of the high-water line, beginning with the most valuable water-fronts. After these were plotted, exterior lines for solid fill and open piers were established, care being taken to keep these lines well inshore from the navigable channel, and where United States Harbor lines were established, to conform thereto. The exterior lines sometimes were fixed by distances from the high-water line, but more generally from dedicated streets or roads above high water.

A riparian owner desiring to buy, or lease, the State's title must furnish a description of his property, evidence of title, and a survey showing the high-water line as of the date of the application. The Board* fixes a price per foot based on upland values. At present, these prices range from a minimum of $1.00 to several hundred dollars per linear foot. If it be leased, the owner pays 7% of the capital sum, semi-annually in advance. If the applicant desires to purchase and the rate fixed be accepted, a deed, or grant, conveying the State's title is prepared, signed by members of the Board, approved by the Governor, the State seal attached, and attested by the Secretary of State.

* In 1915, all the water-front interests of the State of New Jersey were combined in the Board of Commerce and Navigation.

In administering the riparian law, allowance has always been made for reclamation between high and low water prior to 1869 under "common law" rights as well as improvements made in compliance with the Wharf Act.

The revenues received from sales, leases, licenses, etc., to June 30, 1926, are shown in the following tabulation:

Grants and licenses..................	$9 348 719.97
Leases (capital sum)..................	5 625 414.47
Rents and leases to 1926..............	2 480 809.78
Royalties, sand dredging..............	93 505.23
Total...........................	$17 548 449.45

Under the Constitution, the income from this fund is distributed among the public schools of the State.

In fixing the direction the side lines of a grant under water shall take, what is known as the Massachusetts Rule is generally followed; that is, where the shore line is practically straight, the lines shall be at right angles to it. In fixing the side lines in a cove, the length on the exterior line will be shorter than that on the high-water line and a frontage for each owner on the exterior line must be made proportional to his length on the high-water line. Similarly, an owner on a headland will be given a greater frontage on the exterior line than his frontage on the high-water line. No arbitrary rule will fit every case, but the principle of equitable apportionment holds good in all cases. Some Courts have held that where the lands in question are on a navigable river, the side lines shall be at right angles to the thread of the stream.

The position of the high-water line along flat beaches subject to change at every storm, has been the source of much litigation where the lands have become valuable. A land company, as the original owner of the ripa selling such property, should keep an accurate record of the high water, being careful not to sell a parcel intersected by this line if it desires to hold the water-front. Good practice has led to tide-gauge readings over a considerable period to fix the height of a mean high-water line, from which a contour could be run over the desired territory. This is very satisfactory on marsh or other land not subject to erosion and accretion, but in applying this method to a sandy beach where a single storm may lower it a couple of feet, one must do so under average conditions.

Locations of the threads of streams when once determined, should be fixed by courses and distances tied into shore points by closure, ranges, or other method permitting accurate retracement. On narrow streams where the lands are sufficiently valuable to permit accurate work, a base line tied into shore lines should be run and a closure made. From the base, locate both banks carefully; plot them on a scale not less than 200 ft. to an inch, preferably larger; lay courses along the averaged middle, scale the angles and distances, and compute a closure with the land lines and balance by placing errors in the watercourses.

Where a stream boundary has become part of a lake and the stream has become filled with silt, the original outlines may sometimes be retraced by

sounding or wading. Land title companies have made an important contribution by insisting that boundaries must be determined accurately if they are to be insured, and their attitude has helped the land surveyor in educating the public to a better appreciation of the value of good surveying.

The boundary line between New York and New Jersey in the Hudson River, Staten Island Sound, Kill van Kull, New York Bay, and Raritan Bay, has been definitely fixed since 1889 by shore ranges and under the law these must be examined every three years, making such replacements, repairs, etc., as may be necessary to preserve the range lines. If Texas and Oklahoma had fixed the division line in the Red River by a similar process, how much expensive litigation might have been avoided!

Professor Johnston speaks of the improvement in laying out sub-divisions.* Here, again, the influence of the title company has been felt as well as that of the city planner. Cities should insist that sub-divisions conform to a system designed by the City, which must necessarily disregard individual boundaries.

The author has done well to emphasize the need of securing the advice of technically trained men for commissions having to do with the fixing of boundaries.* The Society can be very helpful in this matter and people should pay tribute to the excellent service rendered by members of the profession in fixing indefinite boundaries, by compromise.

It is greatly to be regretted that boundary surveying does not command the same degree of respect given to other branches of engineering. The problems involved in retracing boundaries require a rare and high order of scientific ability and, therefore, should command a recognition equal to that obtained in other fields of engineering. Professor Johnston has rendered a valuable service in calling attention† to the lack of fundamental training for surveying, the apathy of the engineer generally to this branch of the profession, and many other points. It is hoped that his excellent paper may arouse sufficient interest to cause some of the best talent from the colleges to enter this field.

G. M. Bowers,‡ M. Am. Soc. C. E.—Political boundaries are, indeed, too often established without considering data which can only be presented by the engineer or surveyor. A glance at a map showing the corporate boundary of many cities and towns will suffice to convince one of this fact. Whether the line be National, State, or local in its aspect, political and social interest will always outweigh economic interest in the establishment of political boundaries. Scientific considerations, if presented, bear little or no influence in the face of organized political opposition.

Take, for instance, the case of the enlargement of the area of an average sized city through annexation proceedings. Opposition is set up both within and without the existing boundary line; factions form for and against the action. Regardless of the necessity or the expediency of the proposed annexation, boundary lines are tentatively laid down as dictated by political com-

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 2017.
† *Loc. cit.*, p. 2015.
‡ Asst. Director of Public Works, Richmond, Va.

promise and not, by any means, based upon necessities adduced through scientific study.

As long as the engineer or surveyor feels that his work is limited to the duties of "running out" boundary lines described for him, he must rest satisfied when he obtains a reasonable interest in good surveying. Neither can the public be expected to understand him when he advises control surveys to meet the ends only of accuracy and precision in the art of surveying. Should he, on the other hand, consider the future through the experiences of the past, he will have no trouble in being understood either by the politician or the public whose interest he represents.

In almost any community he can easily show, as an argument for careful planning, many existing inconsistencies affecting personal property interest and the unlimited trouble and expense experienced by reason of past inaccuracies.

Too few American cities to-day recognize the value of adequate map information covering, not only the present incorporated area, but its future territorial growth as well. Even less do they understand the prerequisites necessary to obtain accurate map information, such as triangulation or traverse control surveys.

No engineer can dispute the fact that every city or town requires that an accurate standard survey be made if its public function is to be properly and economically administered. It is not saying too much to state that it is the duty of every city or town to not only survey and map its incorporated area, but its surrounding territory as well. The present-day tendency toward decentralization of urban communities, brought on by the advent of the automobile, highway improvements, and land development, is almost universal. Boundaries are soon extended to include this outward growth and, in a few years, the necessity is again at hand for further extension of new boundaries.

The community incorporated is vitally affected by the surrounding developments, which in time will inherit its advantages or disadvantages, and thus become either an asset or liability on those who have to live with it afterward.

Why then should these conditions be allowed to continue as in the past, wherein the community life and progress is so closely related as to form a practical unity, divided only by political lines?

It is fortunate that this question is being recognized in many of the States by their adoption of an act to provide for the recordation of sub-division plats; for the control of construction of public improvements; and for the rights of the incorporated community in or within definite limits of its political boundary.

Can a city or town justify, economically, the expense incurred in surveying and studying the territory beyond its corporate limits with a view toward future annexation and control? Perhaps this question can best be answered in relating the experience of the City of Richmond, Va.

The area of the city, previous to its last annexation in 1914, was approximately 10.75 sq. miles. In that year 13.25 sq. miles were added, making its present total incorporated area approximately 24 sq. miles. In the last, as

well as in the nine separate preceding annexations to the original town established in 1742, the territory annexed had become populated and improved through sub-divisions previously laid out with streets and alleys dedicated for public use, and lots had been sold to individuals, all without consideration of topography, drainage, continuation of thoroughfares, or other essential physical facts. Each extension of the city's corporate area was authorized by Council action, included in which was defined the proposed new corporate line by metes and bounds projected by means of scale from the existing county map. Action by the City Council, authorizing annexation, is with the consent of the General Assembly of Virginia, subject, however, to the approval of the Circuit Court having jurisdiction over the county or counties in which the proposed area lies. The approval of the Circuit Court was granted by decree, on the evidence introduced and on facts disclosed from inspection by the Court of the territory affected.

Following confirmation of its annexation proceedings the City, by Council action in appropriating funds for the purpose, began the surveys of the territory annexed. It is to be noted that these surveys followed, rather than preceded each annexation. Thus were the official maps of the city obtained; each annexation survey producing maps of its limited territory, with varying standards of accuracy due to the methods used in so short a time. Not only did the city suffer through lack of horizontal co-ordination of its map information, but through lack of vertical co-ordination as well, since there were no less than four different level planes in use.

The need for uniform accuracy and map co-ordination was recognized for years, but it was not until 1921 that definite steps were taken to acquire this information. The City Council in that year appropriated $3 000 000 for general sewer construction.

The construction was to cover many sections of the city, principally the outlying portions in which considerable data were then to be collected through surveys before the design of the sewers could be made. Immediately following this, a program of surveys was laid out, which was extended to cover the territory beyond the corporate limits wherein it was necessary to study the drainage situation. Further need for map information beyond the corporate limits was desired at this time on account of the pending Enabling Act granting to the city the approval of plats sub-dividing land, control of the construction of public improvements, and for the rights of cities in or within 10 miles of the corporate limits. Such an act was passed in 1922, but amended in 1924 to read "within five miles of the corporate limits of any city containing more than 150 000 inhabitants".

The City Council after having been presented with the facts, and in appreciation of the needs of the surveys, made an additional specific appropriation for the purpose.

Field surveys were started in 1921 and continued uninterruptedly through 1923. The surveys embraced a scheme of precise triangulation-traverse control covering an area of 60 sq. miles, inclusive of the present incorporated area of 24 sq. miles; a resurvey of 5 sq. miles of the old city for the purpose

of correctly mapping and monumenting this congested and most valued portion of the city; the co-ordination, through geodetic control, of all existing annexation maps of acceptable accuracy; and the topographic mapping of an area of more than 30 sq. miles lying outside the present corporate line. Since 1923 topographic surveys of the outside area have been made with reduced activity on account of lack of sufficient funds. It is the intent of the City to resume, in the near future, topographic surveys of its outlying territory; and for this purpose, the traverse and vertical control is now being extended.

As a result of these surveys, Richmond may be listed with a few other American cities that possess complete and comprehensive geodetic, topographic, and other accurate maps.

Following the completion of the triangulation-traverse control and topographic surveys in 1923, there was published a report* of the result of this work showing why such a survey was needed and giving valuable data for the control of all future surveys, with an outline of the mistakes of the past.

The cost of the 1921-23 surveys was $69 028. The total spent in the territory outside was approximately $52 000, to which has since been added about $13 000, making a total of approximately $65 000 expended to January, 1927, on surveys and map information in securing physical data and in planning the future development of the city in the contiguous unincorporated area.

Since the adoption of the Enabling Act in 1922, the City of the future has acquired, without cost, title of immense value in the large area of lands reserved for streets, alleys, drainage rights, and park purposes. Based on the cost to the City of $600 000 actually expended since 1921 in acquiring land for the opening, extending, and widening of streets for traffic and drainage purposes in the 13.25 sq. miles last annexed, the value accruing to the City in acquisitions made in the area covered by the topographic map of the territory beyond, proportionate to their respective areas, can reasonably be assumed to save the City more than $1 500 000 in the next annexation. The expenditure of $65 000 for topographic surveys has, thus far, returned to the City more than $22 for every dollar spent.

Future political boundaries for cities and towns can and will be established on scientific and economic principles if the art of surveying is utilized and the engineer and not the politician is to be heard.

Robert L' H. Tate,† M. Am. Soc. C. E.—On the boundary survey of 1912 between Costa Rica and Panama, a commission was appointed by the late Chief Justice White, consisting of O. M. Leland, M. Am. Soc. C. E., and the late P. H. Ashmead, F. W. Hodgdon, and J. F. Hayford, Members, Am. Soc. C. E.

The President of France decided that the water divide between the Sixalo and Chirripo Rivers was the boundary between the two countries. Chief Justice White recognized that only a careful survey could locate this divide

* A copy of this report will be sent upon application to the Department of Public Works, Richmond, Va.

† New York, N. Y.

and, through the Commission, sent four parties to Bocas del Toro, Panama, to start the survey.

One party, headed by the writer, commenced surveying at a small place called Grabito and worked toward the Caribbean Sea. The country was covered with a thick jungle growth of trees and vegetation, and a number of negroes had to be employed to cut survey lines. The shore was finally reached at Ponto Mono. The other three parties had finished their sections and had been sent back to the United States.

The remaining party then went to San José, the Capital of Costa Rica, then to Santa Maria, and, finally, started a traverse over the mountains near Buena Vista. Warning had been given that the temperature became low during the nights and, fortunately, plenty of clothing had been provided. It was found that the warning was timely for, at night, water was frozen in the pails, and towels, hung up to dry, were frozen stiff.

It was necessary to determine elevations, run the triangulation survey, and measure angles between the tops of mountain ranges that were along the possible divide and the ridge to the south of the Sixalo River. The party surveyed down the ridge that was thought to be the water divide. Instead of finding a good divide, practically all the ridges ran at right angles, and the party had hard work finding where the divide line hit the saddles between the ridges. Most of the work was really wasted because the party ran survey lines and found the junction of two streams.

Very early in the morning, before the fog came up, a number of photographs of the mountain ridges were taken. There was one view down the Sixalo River; also, a view down the Chirripo River, and a photograph of the area between. When the prints were enlarged to a size of 1½ by 10 ft., the hills were well outlined. They had been located by one of the other parties. Disregarding these photographs, the Costa Rican Government claimed that the ridge between the Sixalo and Chirripo Rivers was not there, and disapproved the finding of the Commission.

E. A. Van Deusen,* Assoc. M. Am. Soc. C. E.—The speaker desires to discuss briefly the subject of aerial surveying in relation to boundary surveys.

A very practical and interesting use to which that art can be put, and is being put, is in connection with transmission lines. The speaker has in mind a 35-mile line that has recently been built by the company with which he is associated. An aerial survey was made of the entire line and covered a strip 2 miles wide.

The advantage of an aerial photographic survey for a transmission line is that a complete and accurate picture is presented of the country to be traversed. In the ordinary topographic survey for a transmission line, several trial lines are run, and a sprinkling of survey data is secured; but many structures and other potential obstacles to the final line are omitted, and data concerning forests and swamps are necessarily incomplete. With the

*Hydr. and Structural Engr., Central Hudson Gas & Elec. Corporation, Poughkeepsie, N. Y

aerial map, on the other hand, each farm and orchard, house and barn, and practically every tree, is correctly located and distinctly visible.

On the very complete map thus obtained and finished to a scale of, say, 600 ft. to the inch, the best feasible location for the tower line can be speedily laid out and right of way can be purchased with the minimum of delay. In the event that certain property owners hold out for excessive prices, the line, with ease and confidence, can be relocated on the aerial map and detoured around their properties.

The speaker will quote no definite figures regarding costs and savings. In fact, exact cost comparisons between these two methods as applied to two different lines, cannot well be made. Suffice it to state, therefore, that this experience was very satisfying and that in the one matter of timber cutting alone, a great saving was secured because of the ability to view at once the whole line and thus to reduce to a minimum this expensive item.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

RE-DESIGNING CATAWBA STATION FOR SERVICE ON A LARGE TRANSMISSION SYSTEM

Discussion*

By H. A. Hageman, M. Am. Soc. C. E.

H. A. Hageman,† M. Am. Soc. C. E. (by letter).‡—The re-development of the power site on the Catawba River at India Hook Shoals 23 years after the original project was put into operation, is an excellent example of the possibilities that exist in modernizing old hydro-electric plants.

It shows the opportunities that are open to the engineer to make use of his skill and experience in creating a new development that will produce power and energy from the water more efficiently than formerly. Many water power developments that have been in operation over a long period of years, have antiquated power-generating equipment that is uneconomical from the standpoint of output and maintenance as compared with present-day hydro-electric machinery. The hydraulic structures of an old development, such as the dam, head-works, water conduits, and power station, may be in good condition but the machinery for transforming the potential energy of the stream into electric current is often inadequate if the maximum amount of power and energy is to be obtained.

The re-development makes a much greater utilization of the stream flow than formerly and permits a large increase in the production of power and energy with considerable gain in efficiency. The increased capacity and energy output is due to the larger storage that has been created, the greater head used, and the higher efficiency of the modern hydro-electric generating and transmitting equipment.

The increase in head from 23 ft. to a maximum of nearly 70 ft. emphasizes the advantage resulting from a condition often encountered on Southern and

* This discussion (of the paper by W. S. Lee, M. Am. Soc. C. E., presented at the meeting of the Power Division, Asheville, N. C., on April 21, 1927, and published in November, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Hydr. Engr., Stone & Webster, Inc., Boston, Mass.

‡ Received by the Secretary, May 5, 1927.

Western rivers, of the absence of extensive construction in the river valleys. Fear of the oft-recurring floods and various other considerations have caused the inhabitants to do most of their building on high ground back from the river and to reserve the river-bank meadows for agricultural purposes. This results in a much lower flowage cost, when a material rise in water level is made, than is usually found on Northern or Eastern streams. Another distinct advantage to water power development on many of the Southern streams is that the railroads, except in the mountains, generally follow the higher land of the divides and cross the river valleys instead of following the river bank, as is usual in the Northern and Eastern States.

During the period between the construction of the old and the new developments described by the author, considerable progress has been made in the art of hydro-electric power generation and in the transmission of electric energy in large quantities. Probably the greatest advance has been made in the design of hydro-electric machinery; more particularly the water-wheel equipment, in the use of direct-connected vertical shaft units, and in the elimination of inefficient mechanical power transmission. It is also true that the improvement in the methods of analyzing stream flow, greater economy in the design of hydro-electric structures, and more efficient power transmission over long distances, have contributed to the progress realized. In view of this statement a comparison of the hydraulic features of the old and the new Catawba developments might be of interest.

The new station contains only half as many units as the old one, while the combined rated capacity is increased from 6 600 kw. to 60 000 kw. The new units are of the single, Francis runner, vertical shaft type, each direct-connected to the generator, while the old units were of the horizontal shaft type with triple Francis runners, each water-wheel unit delivering its power to the generator through a rope-drive. The over-all efficiency of the old units is stated as being 70.7%, while the combined efficiency of the new units is about 87%, an increase of 23 per cent. The generating equipment in the old station required about 0.73 cu. ft. per sec. to produce 1 kw., while in the new development, with the increase in efficiency and in head, the same result is obtained from about 0.20 cu. ft. per sec. under the maximum head, beside the large increase in output due to greater use of storage.

It is interesting to note that valves of the butterfly type, for water control, are used in the inlets to the new units. This type of valve has been used in water power developments for a long time, but it is only within the last few years that it has been improved so that it could be used for large openings without excessive leakage. There is a great diversity in the type of control mechanism used and in the devices adopted to reduce leakage. It is understood that the Duke Power System has several installations of large butterfly valves, using the spring brass flaps attached to the up-stream side of cast-iron wall-frames to secure water-tightness. It would be of considerable interest to know how these have worked out in service, both as regards water-tightness and whether there has been any trouble from material lodging on these flaps. It would also be of interest to know the amount of

power necessary to operate these gates under different conditions of flow, using the tension-rod type of control mechanism. The use of the direct-connected flyballs and of complete individual units in the governor system is typical of present practice.

It is evident that the flood conditions have been very severe on the Catawba River, since although the generator floor of the old plant was placed 28.1 ft. above the normal tail-water level, and 1 ft. higher than the maximum previously recorded flood level, the flood of 1916 submerged the station with a flow 100% greater than the previously recorded maximum flood. This seems a good reason for making the new generator floor level 21.5 ft. higher than the old floor level, or nearly 50 ft. above the normal tail-water level. This excessive flood condition is probably one explanation of the use of the very large flood-gates, 45 ft. long by 30 ft. high, although there may be other reasons for selecting these large gates in place of a greater number of smaller ones.

From the data given in the paper, it is noted that the usable storage has been increased from about 5 000 acre-ft. to about 236 000 acre-ft. This new storage changes the power and energy aspect of the possibilities at this site. Formerly, there was only sufficient pondage available to enable the flow to meet the daily load factor of the station, but now there is a large regulated flow available, which can be used efficiently during periods of drought.

The lack of natural lakes and storage basins on the rivers in the southeastern part of the United States makes all artificial storage especially valuable, and the relative increase in the primary flow of the stream is often much greater than with an equal amount of additional storage developed on rivers with equal drainage areas in the Northern States.

The development and use of the large storage basins must have a considerable effect on the peak floods of the river and must be a valuable asset to all property owners along the lower valley.

It has been stated that large quantities of silt have been carried by the stream during high-water periods before the development of storage reservoirs. Any definite data regarding the lessened quantity of silt carried with the present storage conditions, or the rate of deposition of silt in the various storage basins, would be valuable. It is understood that this river is used for water supply in several instances. There must be considerable benefit from having clearer water.

The growth of the system load has apparently been very rapid, requiring frequent increases in power facilities, and this particular re-development may be taken as a notable achievement of present-day practice.

The tremendous increase in the size of single units that manufacturers will build now, has given a great impetus to re-development of old plants. The use of the present modern reaction and propeller type runners, efficient vertical thrust bearings, improved draft-tubes, and large control gates have all been factors contributing to the construction of more efficient and larger single units. Some of the points mentioned are illustrated by the following hydro-electric developments.

At the Garvin's Falls Plant of the Manchester Traction, Light and Power Company, of Manchester, N. H., on the Merrimack River, the old plant consisted of six horizontal shaft, triple turbine, direct-connected units, under 30-ft. head. These were installed about 1900, and two units had new horizontal shaft runners installed about 1920. In 1924, the four old, 800-kw., horizontal units were removed and two new vertical shaft, propeller type turbines with direct-connected generators were installed. These new units are 3 000 and 4 000 kv-a., respectively. They increased the station capacity 50% with a corresponding increase of only 15% in the quantity of water required. The turbines are equipped with plate-steel Hydraucone draft-tubes, direct-connected governor flyballs, and adjustable lignum vitæ guide-bearings.

In connection with this re-development, the plant at Kelley's Falls, on the Piscataquog River, was built over from a two-unit, horizontal-shaft, rope-drive plant, very similar to the Catawba Plant, to a single unit, automatic, vertical shaft, propeller type, direct-connected plant.

Another instance of the utilization of the power available under a very low head by means of propeller type runners is the Green Island, N. Y., development of Henry Ford and Son, Incorporated. This plant has a normal head of 13 ft., which is considerably reduced during high water, and contains four 2 000-h.p. wheels, having 4-blade runners 156 in. in diameter, in a siphon flume setting.

A recent development containing modern, vertical shaft, Francis turbines, each 25 000 h.p., under 112-ft. head, and direct-connected to alternating-current generators, is the Bartlett's Ferry development of the Columbus Electric and Power Company. These units have elbow type draft-tubes.

An interesting example of the application of a butterfly valve for use as a head-gate to a water-wheel unit will be found in the Conowingo development now under consideration for The Susquehanna Power Company. Each water-wheel unit, which will have a rated capacity of 54 000 h.p. at full gate and 89-ft. head, will be equipped with a vertical shaft, cast-steel butterfly valve, 27 ft. in diameter, provided with specially designed rubber sealing devices. These valves will be the largest ever constructed and will be located directly at the entrance to the scroll-cases. Before selecting this type of valve a careful investigation was made of all types of head-gates with a view to securing the most satisfactory installation from the standpoint of cost and operation. The valves, as installed, will insure the least leakage of any type of gate under the particular conditions. They will be operated from the power-house floor and can be installed or dismantled by the main station cranes. This type of valve has permitted the most economical power house design.

The subject of water power re-development is a vital one at present. Much additional detail could be added, but the foregoing discussion is intended to cover only some of the salient features.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GEORGE THOMAS KEITH, M. Am. Soc. C. E.*

DIED SEPTEMBER 4, 1927.

George Thomas Keith was born on February 4, 1843, at Bridgewater, Mass. He was the only son of Thomas Mitchell and Caroline (Jones) Keith, and a lineal descendant of Captain Miles Standish, John Alden, and Priscilla Mullens.

Mr. Keith was graduated from the Bridgewater State Normal School at the age of eighteen. In the same year, 1861, he enlisted in the Third Massachusetts Volunteers, Company K, and was sent with his regiment to Newbern, S. C. Although he was actively engaged in five great battles of the Civil War, among which were Goldsboro, Kingston, and Whitehall, he suffered no wounds or illness during his entire service. At the close of the war he refused to apply for a pension, but when it was decided that pensions be granted to all Civil War veterans, he finally agreed to accept one.

Following his discharge from the Army, Mr. Keith became a student for three years in the office of the late Thomas Doane, M. Am. Soc. C. E., of Boston, Mass. During this time he was employed as Land Surveyor on the Cape Cod Railroad and the Hoosac Tunnel.

He completed his apprenticeship in 1864, when he became Assistant Engineer on the construction of the Bradford Branch of the Erie Railroad, which position he held for one year. From 1865 to 1868, he was employed, principally, as Engineer in coal mines in the United States Land Company's office, in Bradford, Pa. He then served as Engineer of the Land Department of the Erie Railroad Company, at Hornellsville (now Hornell), N. Y., from 1868 to 1873.

In 1873, Mr. Keith was made Assistant Engineer in charge of construction of the reservoir of the Lawrence Water-Works, at Lawrence, Mass. He held this position until 1876, when he engaged in private practice, with offices at Olean, N. Y., and Bradford, Pa., until 1889. During this period he constructed water-works plants at Newark, Ohio, and at Chester, Pa., and was engaged at Bradford, Pa., on surveys for pipe lines for the transportation of crude oil. He was also employed as Contractor for the water-works at Cuba, N. Y. In 1889, Mr. Keith became a member of the firm of Woltman and Keith, of New York, N. Y., and as such constructed water-works at Tallapoosa, Ga., Henderson, N. C., and elsewhere. His greatest work during this period was, perhaps, the establishment of the water-works in the seven municipalities of Montreal, Que., Canada, all of which were included in the Montreal Water System. From 1896 to 1904, he was engaged on the construction of electric railroads at Olean, N. Y., and Oil City, Pa.

* Memoir prepared from information on file at Society Headquarters.

Ill health then made it necessary for Mr. Keith to retire from active work for a time. When he had recovered sufficiently, he applied for a position on the New York State Barge Canal which was then under construction. The appointments of engineers on this work were made by competitive essays, or themes, submitted to the Barge Canal Commission. Mr. Keith received the highest award for his paper, and consequently, became the first Resident Engineer to be appointed on the Barge Canal construction in 1904. By personal request to the State Commissioner of Highways, he was transferred to the New York State Highway Department, at Olean, in 1907.

He was the author of "Keith's Railroad Tables", and "Functions of Curves", for the "Transit Note Book", published by Keuffel and Esser, which is well known, and is used as a college text.

In 1870, Mr. Keith was married to Evelyn Agnes Moore, who died in March, 1916. He is survived by an only daughter, Mrs. Mildred Evelyn Thynge.

He was a member of the Grand Army of the Republic; the National Geographic Society; and President of the Cattaraugus County Society for the Prevention of Cruelty to Animals. He was also a member of the Olean Country Club and the Olean City Club.

Mr. Keith was elected a Member of the American Society of Civil Engineers on May 4, 1881.

CHARLES HENRY RUST, M. Am. Soc. C. E.*

Died September 22, 1927.

Charles Henry Rust was born at Great Waltham, Essex, England, on December 25, 1852. He was educated at Brentwood Grammar School in the same county and, in 1872, went with his family to Canada.

Soon after his arrival in Canada, Mr. Rust accepted a position on the Engineering Staff which was then engaged on a preliminary survey of the Ontario and Quebec Railway. Five years later, he entered the service of the City of Toronto, Ont., as Rodman on the Engineering Force under the late Frank Shanly, who was then City Engineer.

In 1881, Mr. Rust was appointed to the office of Assistant Engineer by the late R. J. Brough, M. Am. Soc. C. E., City Engineer and Manager of Water-Works, and, in 1883 was made Assistant Engineer in charge of sewers, which position he held until 1891. During this period he also served as Principal Assistant Engineer, and in the spring of 1892, after the resignation of Mr. Granville C. Cunningham, who had been Acting City Engineer, he was made Acting City Engineer until the appointment of the new City Engineer, the late E. H. Keating, M. Am. Soc. C. E., in July of the same year. Mr. Rust was immediately appointed as Deputy City Engineer, and held this office until February, 1898, when, on the resignation of Mr. Keating, he was appointed City Engineer and Manager of the Water-Works.

During his connection with the City of Toronto, Mr. Rust originated and carried out a complete system of main drainage and sewage disposal; a modern filtration plant; a new intake and tunnel for the water-works; numerous

* Memoir prepared by A. C. D. Blanchard, M. Am. Soc. C. E.

important bridges; and many other improvements of lasting benefit to the city. His duties also included reports on many plans, which, although they required much time and painstaking study, were never brought to fruition.

He reported at intervals on water-works and sewerage problems for neighboring municipalities and occasionally acted as an arbitrator. Although his services were frequently in demand as a consultant and expert, his routine duties occupied so much of his time that he was unable to accede to many requests for professional advice.

In 1912, Mr. Rust, having been attracted by the mild climate, and the prospect of the pending construction of a new municipal water supply, resigned the position of City Engineer of Toronto to accept a similar one in Victoria, B. C. While in Victoria he was requested by the Government to report on the Greater Vancouver Sewerage Scheme. At this time he also reported on the Second Narrows Bridge, near Vancouver, B. C.

In 1918, he returned to Toronto under an engagement with the management of the Toronto Street Railway and Toronto Electric Light Company. Later, these organizations were brought under municipal control, and thus Mr. Rust again became identified with the City of Toronto, this time as an official of the Toronto Hydro-Electric System. He retained this position until his death.

In 1887, Mr. Rust was elected as one of the first members of the Canadian Society of Civil Engineers, now the Engineering Institute of Canada. He was greatly interested in its affairs and served in various capacities until in 1911 he was elected President of the Society. In 1902, he was elected President of the American Society of Municipal Improvements. He was also a member of the Executive Committee of the American Water Works Association and, for many years, of the Royal Canadian Yacht Club as well as of the National Club of Toronto.

In 1879, Mr. Rust was married to Alice Preston, who survives him. His kindly and affectionate disposition endeared him to all those who were privileged to know him intimately. His mature judgment, unfailing courtesy, and tactful manner fitted him for the numerous high positions which he held in the Engineering Profession. He was loved and respected by his many business associates.

Mr. Rust was elected a Member of the American Society of Civil Engineers on April 5, 1899, and served as Vice-President in 1913 and 1914.

GODFREY LEWIS SMITH, M. Am. Soc. E. C.*

Died July 2, 1927

Godfrey Lewis Smith was born in San Francisco, Calif., on July 13, 1876, the son of Charles J. J. and Julia B. (Kelley) Smith. He was graduated

* Memoir prepared by E. G. Rogers, Esq., Newport News, Va.

from the San Francisco Public Schools and High School. In September, 1894, he entered the Massachusetts Institute of Technology and remained there as a student, until February, 1899, with the exception of one year when he was ill.

As a boy, Mr. Smith spent many of his free hours in a machine shop owned by his uncle, Lewis Kelley. He was so adaptable that, at the age of fifteen, he was able to operate all the machines in the shop, including firing the boiler and running the steam engine.

On leaving college he entered the employ of the Newport News Shipbuilding and Dry Dock Company, at Newport News, Va., and from February until June, 1899, he was in charge of filing and indexing hull drawings. From June, 1899, to May, 1905, he was engaged in designing and calculating ships' lines and structure, testing materials, estimating the cost of hulls, and surveying of property.

In September of the same year, Mr. Smith was made Engineer in charge of construction, which position he held until March, 1909. During this time he was in charge of the design and construction of one of the largest dry docks then built in the United States. He was appointed Civil Engineer in March, 1909, and in that capacity had charge of all designs and construction of dry docks, piers, bulkheads, trestles, shipways, buildings, tracks, roads, foundations, and other plant work, including some mechanical equipment. He held this position until January, 1924, when he was transferred to the Sales Department of the Company where he remained until his death, at which time he was Acting Sales Manager.

Mr. Smith took a prominent part in the civic and political affairs of Newport News and at one time was President of the Chamber of Commerce and a member of the Board of Directors of that body for a number of years. He was also a member of the Chamber of Commerce of the State of Virginia.

He served several terms as a member of the City Council and during a part of this time he was Chairman of the Finance Committee. He waged an almost constant fight for sound city financing and was an authority on the subject.

For many years before the city management form of government was adopted in Newport News, Mr. Smith was an interested and active advocate of simplified municipal government, and it was largely through his efforts that this plan was accepted by the city. He was so active in behalf of the reform that he was known to many of his associates as the "Father" of the City Manager form of government and when the new method became effective, he was the only member of the old Council to be returned to office.

On June 9, 1908, Mr. Smith was married to Miriam Post who, with four children, Margaret Post, Elizabeth Lewis, Walter Post, and Godfrey Lewis, Jr., survives him.

Mr. Smith was elected a Member of the American Society of Civil Engineers on July 6, 1920.

JAMES ROY MAC BEAN, Assoc. M. Am. Soc. C. E.*

Died July 8, 1927

James Roy MacBean, the son of James Dean and Mary (Murphy) MacBean, was born in Philadelphia, Pa., on January 19, 1885. On the completion of his early education in the public schools, he entered Drexel Institute for a course in Civil Engineering.

In the spring of 1903, Mr. MacBean entered the employ of the Chicago, Rock Island, and Pacific Railway Company as Rodman and Instrumentman and remained with that Company for about three years. Subsequently, he was engaged for about six years on work with the Pennsylvania and Chicago Great Western Railroad Companies. He was employed by the Spanish-American Iron Company in charge of mining and railroad construction in Cuba during parts of 1906 and 1911.

For various periods between 1907 and 1924, Mr. MacBean was engaged for about five years as Draftsman and Chief of Party on sub-division and municipal work with private engineering firms of Philadelphia and Jenkintown, Pa. In 1918, as Assistant Superintendent for the Austin Company, he was engaged on the construction of a concrete roundhouse for the Philadelphia and Reading Railway Company. Later, he accepted a position as Erecting Engineer for the United Gas Improvement Construction Company, of Philadelphia, remaining with this Company until the summer of 1923.

From August, 1924, until his death, Mr. MacBean was employed as Resident Engineer for the New Jersey State Highway Department. As such, he was engaged on reconnaissance surveys in connection with the entrance and connecting roads for the Delaware River Bridge Extension from Camden, N. J., to the main State highway routes of Southern New Jersey. On the completion of the preliminary work he had charge of the construction of the northerly section of the connecting road. Shortly before his death, he had completed the construction of the concrete State Road between Egg Harbor and Mays Landing, in Atlantic County.

Mr. MacBean met his death as the result of an automobile accident while returning from work on the afternoon of July 8, 1927.

He was married on May 16, 1914, to Esther C. Copperfield, of New York, N. Y., who, with a son, Roy Hamilton, survives him. He was a member of the West Park Presbyterian Church, of Philadelphia.

Mr. MacBean was elected an Associate Member of the American Society of Civil Engineers on June 6, 1927.

* Memoir prepared by M. W. Grimes, M. Am. Soc. C. E., and E. H. Maier, Assoc. M. Am. Soc. C. E.

VOL. LIII

SUBJECT INDEX

AUTHOR INDEX

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the Board in this manner.

It is especially urged, in communications concerning applicants, that errors in the record be pointed out and a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from January 1, 1927.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct work	30 years	10 years*	5 years
Associate Member	Qualified to direct work	25 years	6 years*	1 year
Junior	Qualified for sub-professional work	18 years†	2 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers			
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 2 years' active practice.

† Membership ceases at age of 32 unless transferred to higher grade.

LIST OF APPLICANTS.

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RUSSELL, LEWIS W.	Chicago, Ill.	18
SCHEETZ, HARRY L.	Philadelphia, Pa.	18
SERRURIER, MARK U.	Los Angeles, Cal.	18
SHULMAN, MORRIS J.	New York City	18
SINGLETON, CHARLES C.	Baltimore, Md.	18
SMEED, HAROLD F.	Rochester, N. Y.	18
SNYDER, MORRIS K.	Pullman, Wash.	19
SORENSON, JOHN	Omaha, Nebr.	19
SPANGLER, MERLIN G.	Ames, Iowa	19
SUEHRSTEDT, HENRY G.	Hinsdale, Ill.	30
TARTT, PHILLIPS B.	San Antonio, Tex.	30
VAN ANTWERP, EUGENE I.	Detroit, Mich.	19
VANONI, VITO A.	Los Angeles, Cal.	20
VOLKMAR, KARL	Williamsport, Pa.	20
WADDINGTON, JOHN C.	Sheffield, England	30
WAGNER, EDWIN B.	Downingtown, Pa.	20
WARD, VERNON C.	Pittsburgh, Pa.	20
WELLE, EBERHARD	New York City	26
WEST, RANDOLPH C.	Houston, Tex.	31
WESTERGAARD, HARALD M.	Urbana, Ill.	26
WOOD, WALTER J.	Stanford Univ., Cal.	20
YOUNG, CLARENCE R.	Toronto, Canada	26
ZEHNER, JOHN R.	Utica, N. Y.	21

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

The number in the center above each record indicates the serial number of the applicant for the current year, and that at the left the district in which he resides.

The abbreviations in Italics represent, respectively, *TT*, Total Time; *SP*, Sub-Professional Work; *P*, Professional Work; *RC*, Responsible Charge; *D*, Design. The figures showing the amount of time spent in Responsible Charge and on Design are the estimate of the Applicant. The allowance of two years for graduation and of one-third of a year for each academic year successfully completed in an engineering college without graduation is included in Total Time and Sub-Professional Work.

FOR ADMISSION

1

(1) ABILDSO, HAAKON ANDREAS, 1033 Fifty-Seventh St., Brooklyn, N. Y. (Age 27. Born Aker, Oslo, Norway.) 1921 B. S. in C. E., Norway Inst. Tech. *TT 2: SP 2.* Aug. 1921 to Oct. 1922 Asst. Engr., Norwegian Govt., on design and field supervision of transmission towers for high-tension line. *TT 1.2: P 1.2: RC 0.5: D 0.9.* Jan. to April 1923 Draftsman and Surveyor with Browne & Sharpe, Inc., Providence, R. I. *TT 0.3: SP 0.3.* April 1923 to March 1925 Structural Steel Designer, Stone & Webster, Boston, Mass., mostly power stations. *TT 1.9: P 1.9: D 1.5.* March 1925 to date Structural Steel Designer, Union Carbide & Carbon Corporation, New York City, structural steel and reinforced concrete design of buildings for gas and chemical plants. *TT 1.7: P 1.7: D 1.7.* *TT 7.1: SP 2.3: P 4.8: RC 0.5: D 4.1.* Refers to A. P. Andersen, H. C. Borchgrevink, S. R. Donnellon, J. Leahy, R. N. Shepard.

2

(8) ANDERSON, NORVAL EUGENE, 7030 Paxton Ave., Chicago, Ill. (Age 29. Born Anna, Ill.) 1920 B. S. in C. E., Univ. of Ill. *TT 2: SP 2.* June to Sept. of years 1918 and 1919 Engr.'s Asst., Rohrbough Eng. Co., Omaha, Nebr., drafting and surveying. June 1920 to date with San. Dist. of Chicago, until Aug. 1922 as Jun. Asst. Engr., San Div., on drafting, design, computations and minor designs, then Asst. Engr., Sewage Treatment Plant, on design of sewage-treatment works. *TT 6.5: SP 0.2: P 6.3: RC 2: D 4.3.* *TT 8.5: SP 2.2: P 6.3: RC 2: D 4.3.* Refers to L. B. Barker, O. L. Eltinge, I. P. Kane, H. L. McMillan, L. Pearse, H. P. Ramey, H. S. Ripley, L. C. Whittemore.

3

(1) BARBATO, THEODORE, 154 Nassau St., New York City. (Age 38. Born in Argentine Republic.) 1910 B. E., and 1914 C. E., Cooper Union Inst. *TT 2: SP 2.* 1910 to 1917 with Public Service Comm., 1st Dist., New York City, until 1914 as Draftsman and Asst. Engr., designing subway and elevated railroad structures, then Asst. Engr. in charge of Inspection Party on Third Ave. Elevated structure, New York City. *TT 6.5: SP 1: P 5.5: RC 4.5.* 1917 to 1918 Designer, Perin & Marshall, New York City, on structural steel industrial buildings and power houses. *TT 1: P 1: RC 1.* 1918 to 1919 Designer, H. D. Best Co., New York City, on reinforced concrete industrial buildings. *TT 1.5: P 1.5: RC 1.5.* 1919 to 1920 Designer, Concrete Steel Eng. Co., New York City, designing and detailing reinforced concrete bridges. *TT 1: P 1: RC 1.* 1920 to 1923 Chf. Engr., Schuster Eng. Co., and 1923 to date Chf. Engr., Jno. T. McCoy, Inc., New York City, designing, estimating, and supervising design and construction of reinforced concrete and structural steel industrial and commercial buildings. *TT 6: P 6: D 6.* *TT 18: SP 3: P 15: RC 8: D 6.* Refers to E. H. Harder, C. W. Hudson, G. M. Purver, W. Mueser, E. Wegmann.

4

(4) BAROFSKY, MAX, 700 South Sixteenth St., Philadelphia, Pa. (Age 27. Born Philadelphia, Pa.) 1921 B. S. in C. E., Univ. of Pa. *TT 2: SP 2.* Jan. 1922 to date with City of Philadelphia, until March 1925 as Structural Draftsman, Bridge Div., Bureau of Eng., on complete design of highway bridges, mainly concrete arches, and since then Asst Engr., Bridge and Sewer Div., Bureau of Highways, on investigation of bridges, making reports